FATIGUE AND STATIC BEHAVIOUR OF POST-TENSIONED STEEL- CONCRETE COMPOSITE BEAMS

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STEEL- CONCRETE COMPOSITE BEAMS

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opinion, it is worthy of acceptance.

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Dedication

I dedicate this dissertation to my loving father and mother, my supportive wife, and my sweet kids. Their love, encouragement, and prayers enabled me to accomplish such work. I owe them an enormous debt of gratitude for being my source of joy through the trials and struggles of my Ph.D. journey.
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# Table of Contents

Acknowledgements ............................................................................................................. ii
List of Figures ...................................................................................................................... v
List of Tables ..................................................................................................................... ix
Abstract ................................................................................................................................ x

Chapter 1: Introduction ................................................................................................... 1
  1.1 Background ....................................................................................................... 1
  1.2 Goal and Objectives .......................................................................................... 4
  1.3 Dissertation Structure ....................................................................................... 5
  1.4 References ........................................................................................................ 7

Chapter 2: Post-Tensioning Effect on Fatigue Behavior of Damaged Steel-Concrete Composite Beams ............................................................................................................ 8
  2.1 Abstract ............................................................................................................. 8
  2.2 Introduction ...................................................................................................... 9
  2.3 Objectives ....................................................................................................... 11
  2.4 Experimental program .................................................................................... 12
  2.5 Procedures ...................................................................................................... 16
  2.6 Results and discussions .................................................................................. 18
  2.7 Summary and Conclusion ............................................................................... 31
  2.8 References ...................................................................................................... 34

Chapter 3: Pre-Damage Effect on the Residual Behavior of Externally Post-tensioned Fatigued Steel-Concrete Composite Beams ............................................................ 37
  3.1 Abstract ........................................................................................................... 37
  3.2 Introduction .................................................................................................... 38
  3.3 Experimental program .................................................................................... 42
  3.4 Results and discussion .................................................................................... 48
  3.5 Summary and Conclusions ............................................................................. 59
  3.6 References ...................................................................................................... 62

Chapter 4: Numerical Investigation of the Monotonic Behavior of Strengthened Steel-Concrete Composite Girders ......................................................................................... 65
  4.1 Abstract ........................................................................................................... 65
List of Figures

Fig.1. 1  Schematics showing cross-section of typical components of a strengthened SCC beams..........................................................2

Fig.1. 2  Typical components of a strengthened SCC beams in the longitudinal direction. ..............................................................................................................................................2

Fig. 2. 1  Samples’ cross-section. (dimensions shown are in mm)..........................24

Fig. 2. 2  Loading setup and details of the locations of the composite section components. (dimensions shown are in mm).................................................................15

Fig. 2. 3  Instrumentation devices......................................................................................19

Fig. 2. 4  AASHTO design S-N curve for fatigue capacity of the headed shear studs….20

Fig. 2. 5  Testing procedures. Stages F: Fatigue, P: Post-tensioning, E: Environmental changes, S: Static deformation.................................................................21

Fig. 2. 6  Outdoor environmental changes...........................................................................22

Fig. 2. 7  Typical hairline crack in the concrete slab (a) side (b) top surface............23

Fig. 2. 8  Crack patterns of non-damaged samples. (Numbers of cycles shown next to cracks are in thousands and indicate the initiation of cracks)..................................24

Fig. 2. 9  Crack patterns of pre-damaged samples. (Numbers of cycles shown next to cracks are in thousands and indicate the initiation of cracks)..................................26

Fig. 2. 10 Incremental strains in the shear connectors near supports.........................28

Fig. 2. 11 Midspan incremental deflections.................................................................30

Fig. 2. 12 Incremental interface slippage........................................................................30

Fig. 2. 13 Compressive strains in the deck.  Fig. 2. 14 Tensile strains in the steel flange.........................................................................................................................32
Fig. 2.15 Incremental PT force during the fatigue tests for the strengthened samples. 32

Fig. 3.1 Dimensions and reinforcement details for (a) Cross-section and (b) Elevation. 45

Fig. 3.2 (a) Instrumented devices locations (b) Regular & surface mounted SG (c) Load cells 48

Fig. 3.3 Different types of the pre-damage applied to the tested samples 50

Fig. 3.4 Loading setup; (a) Fatigue test setup and (b) Static test setup 51

Fig. 3.5 Crack patterns of the concrete decks before the final static testing to failure 56

Fig. 3.6 Pre-damage effect on (a) Load-deflection relationships, and (b) Slippage 57

Fig. 3.7 Pre-damage effect on the shear connectors’ strains 58

Fig. 3.8 Residual strains in the (a) Bottom flange of the steel beams, and (b) Top surface of the concrete decks 59

Fig. 3.9 The typical failure mode after the completion of the residual static tests 60

Fig. 4.1 Finite element model; loading setup (top), meshing (bottom left), and details of the different components of the composite beam (bottom right) 73

Fig. 4.2 Cross-section geometry 74

Fig. 4.3 Locations of the components of the composite beam (Dimensions are in mm). 74

Fig. 4.4 Comparison between numerical and experimental results; a) Load-deflection, b) Near end connector strains, and c) Midspan concrete strains 76
Fig. 4.5  Failure modes of experimental and FE model tests. a) concrete crushing (Experimental) b) concrete crushing (FE model) c) steel yielding (Experimental) d) steel yielding (FE model) ..............................................................................................................................................78

Fig. 4.6  Effect of degree of shear connection; a) Load-deflection, b) Connector strain, c) Slippage, d) Tendon force ............................................................................................................................................81

Fig. 4.7  Top view of the steel beams showing the distribution of the studs .........................................................................................82

Fig. 4.8  Effect of stud connector arrangement (1R vs. 2R) and diameter d (in mm) on a) Deflection, b) Slippage and c) Tendon force ........................................................................................................................................82

Fig. 4.9  Effect of initial post-tensioning force; a) Load-deflection, b) Deflection-steel strain, c) Load-steel strain, and d) Load-tendon force ..........................................................................................................................86

Fig. 4.10  Schematics showing the varying dimensions ...............................................................................................................................86

Fig. 4.11  Effect of steel beam size; a) Load-deflection, b) Steel strain-deflection c) Load-steel strain, and d) Load-tendon force ..........................................................................................................................89

Fig. 4.12  Effect of concrete compressive strength; a) Deflection, b) Slippage, and c) Tendon force .................................................................................................................................................90

Fig. 4.13  Effect of degradation of stud connector; a) Deflection, b) Steel strain, c) Tendon force .........................................................................................................................................................92

Fig. 5.1  The loading setup and details of the different components of the composite beam a) longitudinal direction b) transverse direction showing the deck reinforcement .............................................................................................................................................104

Fig. 5.2  Labels for dimensions of the beam cross-section provided in Table 1 .................................................................................................106

Fig. 5.3  The FE and Experimental results for the load-deflection curves for a) sample B1 b) sample B2 and c) sample B3 .............................................................................................................................................107
Fig. 5. 4  Top view of the steel beam showing the numbering of the shear stud rows.
........................................................................................................................................................................109

Fig. 5. 5  Top view of the steel beam showing examples of removed rows: (a) Step 1, one row removed; (b) Step 2, two rows removed...............................................................109

Fig. 5. 6  Slippage example in the steel-concrete interface region at the beam end....110

Fig. 5. 7  Effect of progressive failure on the interface slippage; (a) removal side (b) non-removal side..............................................................................................................111

Fig. 5. 8  Stud fracture effect on the maximum shear force ........................................112

Fig. 5. 9  Stud fracture effect on the minimum shear force........................................113

Fig. 5. 10 Shear stress range across the composite beam after removal of studs. ....114

Fig. 5. 11 Comparison between the theoretical and the FE results ..........................117

Fig. 5. 12 Effect of progressive failure on the midspan tensile strain  ......................119

Fig. 5. 13 The maximum load-carrying capacity as the number of removed studs increases..........................................................................................................................119

Fig. 5. 14 Ratio of maximum load of the damaged beam to that of a beam with no studs removed ..............................................................120

Fig. 5. 15 Load-deflection response (a) and increase in shear force-deflection response (b) for S and NS samples after steps 0, 5, and 10. .......................................................121
List of Tables

Table 2.1  Materials properties for each component of the SCC beam .................16
Table 2.2  Test matrix for the tested samples .................................................................18
Table 2.3  Comparison of the residual maximum loads and deflections .................31
Table 3.1  Summary of the loading scheme .................................................................54
Table 3.2  Details of the naming convention system ...................................................46
Table 3.3  Variations in the ultimate load capacities and midspan deflections ..........58
Table 4.1  Material properties used in both the experimental and the numerical tests ..............................................................................................................................76
Table 4.2  Number of studs and spacings for different arrangement types and diameters .................................................................................................................86
Table 4.3  Test matrix for investigating the effect of the steel beam depth and tendon eccentricity ........................................................................................................91
Table 5.1  Dimensions of the cross-sections of the three beams .................................110
Table 5.2  Details of the different components of the composite girders .................106
FATIGUE AND STATIC BEHAVIOUR OF POST-TENSIONED STEEL-CONCRETE COMPOSITE BEAMS

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Abstract

Strengthening steel and concrete composite (SCC) bridge superstructures using post-tensioned steel tendons offers many advantages. Since bridge structures are usually exposed to fatigue cycles, understanding the fatigue behavior of strengthened composite beams is vital. Also, bridges in practice are subject to various types of damages. Thus, when strengthening an in-service bridge component, the effect of pre-fatigue conditions on the effectiveness of post-tensioned steel tendons should be considered. In this research, the fatigue and static behavior of damaged and undamaged post-tensioned steel-concrete composite beams were evaluated experimentally and numerically.

Experimental fatigue tests were conducted on five SCC samples under four-point fatigue loading. The effect of external post-tensioning (PT) as a retrofit on the cyclic crack patterns on the concrete flange, cyclic incremental deformations, and strains were investigated with various pre-fatigue conditions. The pre-fatigue conditions included exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damages. The results of these experimental fatigue tests showed that external PT enhanced the
performance of the individual components of the composite samples (steel beams, concrete flanges, and shear connectors), which enhanced the overall fatigue performance of the strengthened samples. However, the strengthened samples experienced longitudinal fatigue cracks in the concrete flanges because of PT while such cracks were not observed in the non-strengthened sample. The pre-damage conditions due to fatigue loading or environmental changes caused higher damages in concrete around the studs relative to the plastically pre-deformed strengthened sample and led to more incremental deformations and strains.

Experimental static tests to failure were performed on the fatigued samples to investigate their residual capacities, deformations, and strains. The results of the static testing program showed that the crack patterns in the concrete deck were significantly affected by the type of pre-damage that was applied before the post-tensioning. The static overloading pre-damage reduced the number of cracks and their rate of propagation while the exposure to outdoor environment pre-damage induced more cracks, which negatively affected the crack patterns during fatigue loading. Subjecting the sample to the pre-damage of plastic deformation slightly improved its performance in terms of residual stiffness and ultimate load. The residual ultimate load was increased by 7% relative to the fatigued sample without pre-damage. However, the ductility was reduced by 40% due to the initial plastic deformation. This reduction in ductility was accompanied with a decrease in the interface slippage between the concrete deck and steel flange.

Numerical investigations were performed to study the monotonic behavior of externally post-tensioned steel-concrete composite girders. A three-dimensional numerical model
was developed, which was validated using the experimental test results presented in this research as well as experimental test results that are available in the literature. A parametric study using this validated numerical model was performed to investigate the effects of various parameters on the monotonic performance of composite girders strengthened with external post-tensioned tendons. The parameters investigated include variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the studs. The numerical model provided a better understanding of the effect of these parameters on the behavior of the strengthened beams. The results of the parametric study show that the slippage between the concrete deck and steel beam increased as the degree of shear connection decreased. Also, as the shear connection degree decreased, its effect on the slippage behavior increased. The study also shows that, for the same degree of shear connection, beams with one row of shear studs had higher flexural capacity than beams with two rows of studs. The load-deflection and slippage behavior improved when smaller diameters of the studs were used. Also, The higher the post-tensioning force the higher the ultimate load capacity and the lower the tensile strains in the steel beam. The tensile strains at midspan were considerably reduced by increasing the depth of the steel beam. The lowest midspan tensile strains were obtained from the combination of increasing the depth of the steel beam and post-tensioning tendon eccentricity.

Furthermore, numerical investigations were performed to study the behavior of damaged strengthened steel-concrete composite girders. Loads throughout the service life of bridges may cause failure in the form of fracture in the studs near the ends of the bridge girders.
The effect of this stud failure on the residual static capacity and the residual fatigue life of composite girders is not well investigated in the literature. Therefore, this research presents a numerical investigation on the residual behavior of prestressed composite beams with fractured studs at the end regions of the beams. The objective of this study is to investigate the effects of the progressive failure of stud shear connectors on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. Also, the effect of stud fracture on the slippage, shear stress range and compressive and tensile strains were studied. The behavior of the composite girders in terms of the estimated fatigue life and residual capacity was affected by the number of removed studs. Until 15% of the rows were removed, the strengthened sample had a better response in terms of the tensile and compressive strains and residual ultimate load. The effect of stud fracture on the shear stress ranges experienced by the shear connectors mainly manifested at the beam ends where stud fracture occurred. Also, neglecting the steel-concrete interface slippages in the theoretical calculation of stud shear stress ranges resulted in a significant underestimation of the shear stress ranges experienced by the shear connectors.

In summary, in this research, the effect of external PT and the effects of pre-existence of damage on the performance of the different components of the steel-concrete composite section during fatigue were experimentally investigated. The residual capacities of damaged and non-damaged fatigued steel-concrete composite beams strengthened with externally post-tensioned steel tendons were also experimentally tested. Furthermore, a three-dimensional finite element (FE) model that can simulate the behavior of composite steel-concrete girders strengthened with external post-tensioned steel tendons was developed and validated. Then, numerical parametric studies were performed using the
validated FE model to investigate the effects of various important parameters on the flexural behavior of the strengthened composite girders. Also, the effect of progressive stud fracture on the behavior of strengthened SCC girders was investigated.
Chapter 1: Introduction

1.1 Background

Steel-concrete composite (SCC) beams are practical and cost-effective choices for highway bridges. The advantageous strength of steel in tension is combined with the compressive strength of concrete to produce sections that are stronger and more economical compared to using either material alone. The steel beam is connected to the concrete deck via shear connectors to transfer the shear and prevent slippage between the top of the steel beam and the bottom of the concrete deck. The shear connectors play a vital role in increasing the static and fatigue strength of the SCC girders (Sutter 1966; Lorenc and Kubica 2006). The static and fatigue strengths are ensured when full composite action is created by ensuring that sufficient shear connectors are provided (Ovuoba and Prinz 2016).

Repeated loads of highway traffic on SCC members often cause gradual strength degradation of the different components of the composite section (Murgudkar and Joshi 2017). Typical components of the composite section are shown in Fig.1.1 and Fig.1.2. A high number of the repeated fatigue cycles usually result in fatigue cracks in the different components of the SCC girders such as the concrete deck, steel beam, and shear connectors. Damages caused by fatigue or other deterioration mechanisms can be avoided, minimized, or repaired by using external post-tensioning, which is commonly used in SCC highway bridges as a strengthening method. Typically, the strengthening is done using high-strength steel tendons positioned near the tension fiber of the SCC girder.
The flexural performance of the SCC girders is largely affected by the performance of its individual components, such as the steel beam, concrete slab, slab reinforcement, and shear connectors. The behavior of externally post-tensioned SCC beams under fatigue loading is not well understood as little research exists in this field. Most of the previous fatigue studies were conducted on non-strengthened SCC beams or by using standard pushout tests. While pushout tests provided valuable information, they do not represent the actual stress state in large-scale composite beams.

![Fig.1. 1 Schematics showing a cross-section of typical components of strengthened SCC beams.](image1)

Furthermore, the PT force cannot be applied in pushout testing setups. The external PT is mostly used for in-service bridges as a repairing method. Therefore, one of the main objectives of this study is to experimentally investigate the effect of external PT on
deformations and strains of the different components of the composite section during fatigue with various pre-fatigue conditions. The pre-fatigue conditions investigated in this paper include exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damage.

In the existing literature, no studies were found that address the residual behavior of damaged prefatigued SCC beams that were externally strengthened with post-tensioning tendons. Existing literature related to the strengthening of existing bridges focused on reinforced concrete bridge girders or SCC beams that are strengthened with methods other than steel tendons such as carbon fiber reinforced polymers. Also, the current studies mostly focused on the monotonic behavior of bridge members without considering the effect of undergoing fatigue cycles. Furthermore, the effects of possible pre-damages on the efficacy of strengthening of SCC beams were not studied. Thus, among the objectives of the present study is to expand the state of knowledge by evaluating the effects of the pre-existence of damage on the performance of strengthened SCC beams. The evaluation was carried out experimentally by comparing the residual performance of damaged and non-damaged fatigued steel-concrete composite beams strengthened with externally post-tensioned steel tendons.

Due to the cost and time-consuming nature of experimental studies, studied parameters are usually limited to a few parameters related to the behavior of SCC beams. Thus, among the objectives of this study is to develop a three-dimensional finite element model that can simulate the behavior of composite steel-concrete girders strengthened with external post-tensioned steel tendons. The model, validated using the results of experimental tests, will
be used to perform parametric studies to investigate the effects of various important parameters on the flexural behavior of damaged and undamaged strengthened steel-concrete composite girders.

1.2 Goal and Objectives

The goal of this study is to evaluate the fatigue and static behavior of damaged and undamaged strengthened steel-concrete composite beams. The main objectives of the study are listed below:

- Experimentally investigate the effect of external PT on the performance of the different components of the steel-concrete composite section during fatigue.
- Expand the state of knowledge by evaluating the effects of pre-existence of damage on the performance of strengthened SCC beams.
- Evaluate the residual capacities of damaged and non-damaged fatigued steel-concrete composite beams strengthened with externally post-tensioned steel tendons.
- Develop a three-dimensional finite element (FE) model that can simulate the behavior of composite steel-concrete girders strengthened with external post-tensioned steel tendons.
- Perform parametric study using the FE model to investigate the effects of various important parameters on the flexural behavior of the strengthened composite girders.
- Numerically study the effect of fracture of shear connectors on the behavior of strengthened SCC beams.
1.3   **Dissertation Structure**

This dissertation is divided into chapters as follows:

**Chapter 1** gives a brief introduction to the research and states the research’s goals and objectives. It briefly discusses the problem statement and the need for this research. Detailed review of the literature is provided in each of the subsequent chapters.

**Chapter 2** is an experimental investigation on the fatigue behavior of strengthened steel-concrete composite beams. In this chapter, five SCC samples were experimentally tested under four-point fatigue loading. The effect of the external post-tensioning as a retrofitting technique on the cyclic crack patterns on the concrete flanges, cyclic incremental deformations, and cyclic incremental strains were investigated with various pre-fatigue conditions. Three of the tested samples were subjected to outdoor environmental changes, cyclic preloading, and static overloading as pre-damages before applying the external post-tensioning. Also, non-damaged strengthened and non-strengthened samples were tested as references to those with pre-damages.

**Chapter 3** is an experimental investigation to investigate the static residual capacities of fatigued steel-concrete composite beams. This chapter discusses the results of the experimental residual static tests that were conducted on the fatigued samples after the completion of the fatigue testing program of Chapter 2.

**Chapter 4** is a numerical investigation of the flexural behavior of steel-concrete composite beams. A three-dimensional numerical model was developed and validated using experimental test results that were conducted by the authors. A parametric study using this
validated numerical model was performed to investigate the effects of various parameters on the monotonic performance of composite girders strengthened with external post-tensioned tendons. The parameters investigated include variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the studs.

**Chapter 5** presents a numerical investigation on the residual behavior of prestressed composite beams with fractured studs at the end regions of the composite beams. The study was done using a finite element model that was validated by existing experimental work. The objective of this study is to investigate the effects of the progressive failure of stud shear connectors on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. Also, the effect of stud fracture on the slippage, shear stress range and compressive and tensile strains were studied.

**Chapter 6** presents the overall conclusions drawn from the results of this research and highlights its main contributions. Detailed conclusions specific to each of the research objectives are presented as part of each of the chapters. Recommendations for future work are also presented in this chapter.
1.4 References


R. G. Sutter, “The Fatigue strength of Shear Connectors in Steel-Concrete Composite Beams,” no. 316.9, 1966.

Chapter 2: Post-Tensioning Effect on Fatigue Behavior of Damaged Steel-Concrete Composite Beams

2.1 Abstract

Strengthening steel and concrete composite (SCC) bridge superstructures using post-tensioned steel tendons offers many advantages. Since bridge structures are usually exposed to fatigue cycles, understanding the fatigue behavior of strengthened composite beams is very important and the effect of pre-fatigue conditions on the efficacy of post-tensioned steel tendons should be considered. In this paper, five SCC samples were experimentally tested under four-point fatigue loading. The effect of the external post-tensioning (PT) as a retrofitting technique on the cyclic crack patterns on the concrete flanges, cyclic incremental deformations, and cyclic incremental strains were investigated with various pre-fatigue conditions. The pre-fatigue conditions included exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damages. The results showed that the external PT enhanced the performance of the individual components of the composite samples (steel beams, concrete flanges, and shear connectors), which enhanced the overall fatigue performance of the strengthened samples. However, the strengthened samples experienced longitudinal fatigue cracks in the concrete flanges because of PT while such cracks were not observed in the non-strengthened sample. The pre-damage conditions due to fatigue loading or environmental changes caused higher damages in concrete around the studs relative to the plastically pre-deformed strengthened sample and led to more incremental deformations and strains.
2.2 Introduction

Steel and concrete composite (SCC) bridge superstructures offer many advantages over other types of superstructures. Combining the strength of the steel girder and concrete deck members produces a much stronger construction material than steel or concrete alone. This type of construction is extensively used in bridge structures under repeated application of fatigue cycles due to its relatively high stiffness for applied loads (Slutter 1966). To join the concrete deck and the steel beam, headed stud shear connectors are commonly used. This key component provides the shear connection between the two members. Assuring shear connection allows the composite action to take place, which contributes to shear transfer and prevents uplift (Vasdravellis and Uy 2014) (Selvi 2016) (Ovuoba and Prinz 2016).

Bridge structures typically undergo fatigue cycles. Thus, consideration of fatigue is essential as it can be detrimental to their structural performance. The performance of composite bridge members subjected to fatigue has been investigated by many. The main objective for most of these studies was to investigate the behavior of shear connectors under fatigue. The standard push-out tests were commonly used. Slutter and Fisher (1966) were the first to examine the fatigue strength of shear connectors back in 1966. They observed that the most common type of failure was a crack initiating at the reinforcement of the shear connector weld and penetrating the concrete deck. They also proposed an equation to calculate the fatigue life of shear connectors based on the S-N method. The current AASHTO LRFD Bridge Design Specifications (AASHTO 2012) provisions for the fatigue design of shear studs are largely based on their research. Others reported the
fracture of the stud root as another observed failure mode (Xu et al. 2018) (Yu-Hang et al. 2014). Also, the reliability of the stud design models and methods adopted by AASHTO was investigated by (Ovuoba and Prinz 2016) (Johnson 2000).

Post-tensioning (PT) has been used as a strengthening or repairing method for in-service bridges since the 1950s (Klaiber et al. 1988). More recently, the static capacity of SCC beams externally strengthened with post-tensioned tendons has been evaluated by Lorenc and Kubica (Lorenc and Kubica 2006). Six SCC beams were subjected to static loading tests. The effect of external PT and arrangements of tendons (straight or draped) on SCC beams were evaluated. The results showed that external PT enhanced the yield and ultimate strengths and that the tendons’ arrangement did not significantly change the response of these composite specimens. El-Zohairy and Salim (El-Zohairy and Salim 2017) introduced a parametric study that indicated an improvement in the composite beams’ fatigue performance due to post-tensioning and by using longer tendons.

External PT can be used to improve the performance of SCC beams under fatigue (El-Zohairy et al. 2019). Although many researchers investigated the fatigue performance of SCC beams, research on the effect of external PT on such beams is rare. Albrecht et al. experimentally investigated the fatigue strength of 11 SCC beams prestressed with steel tendons with a cover plate welded to the tension flange (Albrecht, Li, and Saadatmanesh 1995). The stud connectors and the cover plates exhibited increased fatigue strength due to using strengthening steel tendons. In 2008, Albrecht and Lenwari have explored several methods of repairing cracked steel beams in SCC girders (Albrecht and Lenwari 2008). Among these methods is keeping the crack intact and using sufficient prestressing to
eliminate tension, caused by the live load, in the cracked steel beam and subject the beam only to compression-compression cycles. For both intact and cracked beams, they reported that full prestressing results in composite beams that are not prone to fatigue damage. Allawi (Allawi 2017) tested two composite concrete girders strengthened with external PT and compared them to two identical girders that were not strengthened. The behavior of the girders was investigated under static and repeated loadings. The results showed that the tendons enhanced the structural behavior of these beams under both loading types. El-Zohairy et al. performed experimental static and fatigue tests on SCC beams with external PT (El-Zohairy, Salim, and Saucier 2019). In this study, the flexural, as well as the fatigue behavior, was improved by adding the post-tensioned tendons. Adding external PT increased the composite beam’s ultimate strength and reduced the tensile strain in the steel beam. Furthermore, it decreased the strains in different components of the composite beams at all stages of fatigue loading.

2.3 Objectives

The behavior of externally post-tensioned SCC beams under fatigue loading is not well understood as little research exists in this field. Most of the previous fatigue studies were conducted on non-strengthened SCC beams or by using standard pushout tests. While pushout tests provided valuable information, they do not represent the actual stress state in the composite beams. Furthermore, the PT force cannot be applied. The external PT is mostly used for in-service bridges as a repairing method. Therefore, the main objective of this study is to experimentally investigate the effect of external PT on deformations and strains of the different components of the composite section during fatigue with various
pre-fatigue conditions. The pre-fatigue conditions investigated in this paper include exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damages.

2.4 Experimental program

Five SCC samples were experimentally tested under fatigue loading. All samples had the same fatigue loading range, tendons’ initial stress, and the number of cycles. To evaluate the effect of PT, one non-strengthened sample was used as a reference. Also, to examine the effect of the pre-fatigue conditions on the behavior of strengthened samples, three of the samples were exposed to various damages before being post-tensioned and then subjected to fatigue loading. The samples’ geometry, applied stress range, and tendons’ initial PT force were chosen following AASHTO LRFD Bridge Design Specifications (AASHTO 2012). The fatigue loading was conducted under a four-point bending test scheme and was set to complete one million fatigue cycles. The effect of the external PT and pre-fatigue conditions were examined by monitoring the fatigue crack patterns on the concrete flanges; cyclic deflections and slippages; and cyclic strains in concrete decks, steel beams, and shear connectors.

2.4.1 Tested samples

The overall span length and loading setup of the samples were selected based on the space availability and equipment capacity in the testing laboratory. Based on the selected span, the overall depth of the steel beams was selected according to the minimum span-to-depth ratios recommended by AASHTO guidelines (AASHTO 2012). The effective width of the
concrete flange was also calculated based on the span length as suggested by AASHTO. The dimensions and reinforcement of the concrete slab were designed to prevent premature failure in the concrete deck such as shear cone failure and longitudinal shear failure modes. The size and properties of the steel beam and stiffeners were selected such that shear buckling failure and crippling of the web in the steel beam are avoided. All samples were 4572 mm long with a simply supported span of 4420 mm in length. The thickness of the concrete flange was 102 mm. The width of the concrete flange was 1143 mm. Fig.2.1 shows the details of the samples’ cross-section geometry. The reinforcement in the concrete flange for all samples was as follows: 12 bars with a diameter of 10 mm spaced at 216 mm in the transverse direction, 38 stirrups with 10 mm diameter spaced 122.2 mm apart in the longitudinal direction. The longitudinal and transverse spacings, height-to-diameter ratio, and the number of studs were selected according to AASHTO’s requirements for fatigue and strength limit states and to ensure full composite action (AASHTO 2012). Thus, two headed stud shear connectors per row were welded to the top flange of the steel beam with a total number of 74 studs. The studs were 15.875 mm in diameter and 81 mm in height with uniform longitudinal spacing between the stud shear connectors of 122.2 mm (see Fig.2.1).

The external PT was applied by using two 7ϕ4.23 mm steel strands. The strands were positioned 32 mm from the bottom of the steel beams and inserted through one end of the sample and extended to the opposite end. These strands were subjected to an initial PT force of 85 kN (46% of their ultimate strength). Each steel beam was stiffened with four stiffeners located at the supports and loading points. Also, head plates were welded to the steel beam ends to help to anchor the post-tensioned tendons. Fig.2.2 shows the locations
and details of the stiffeners and tendons. The steel load plate dimensions were 229x178 mm at load locations and 305x203 mm at support locations.

### 2.4.2 Material properties

The materials properties for each component of the SCC samples are listed in Table 2.1. The concrete compressive strength for all samples was 33 Mpa.

### 2.4.3 Testing setup

The samples were tested under a four-point bending test scheme with a distance between the supports of 4420 mm. The load was applied using a servo-hydraulic actuator with a single jack and a maximum capacity of 890 kN. To transfer the load to the two load points, a spreader beam was used as shown in Fig. 2.2. The loading points were positioned near the thirds of the supported span with a distance between the points of 1372 mm. A sine wave cyclic force was applied with a 4 cycles/second frequency.

![Fig. 2.1 Samples’ cross-section. (dimensions shown are in mm)](image)
2.4.4 Instrumentation

To monitor the tensile and compressive stud strains, eight strain gauges were mounted to four shear studs such that each of the instrumented studs has a strain gauge on both sides (see Fig. 2.3). The locations of the instrumented studs were chosen to be near supports and quarter spans in each sample, as shown in Fig. 2.3. Also, strains at the top surface of the concrete flanges, as well as the bottom surface of the steel flanges, were monitored by attaching strain gauges at midspans and one-third of the supported spans. Linear variable differential transducers (LVDT) were instrumented beneath the samples at midspans to monitor the vertical deflections at all stages of the fatigue loading.

Fig. 2.2 Loading setup and details of the locations of the composite section components. (dimensions shown are in mm)
Table 2.1 Materials properties for each component of the SCC beam

<table>
<thead>
<tr>
<th>Property</th>
<th>Yield strength (Mpa)</th>
<th>Tensile strength (Mpa)</th>
<th>Tensile Modulus (Gpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel beam (A992)</td>
<td>345</td>
<td>450</td>
<td>204</td>
</tr>
<tr>
<td>Reinforcement bars</td>
<td>294</td>
<td>403</td>
<td>208</td>
</tr>
<tr>
<td>Shear studs</td>
<td>352</td>
<td>448</td>
<td>207</td>
</tr>
<tr>
<td>PT tendons</td>
<td>1680</td>
<td>1860</td>
<td>200</td>
</tr>
</tbody>
</table>

The slips in the interface between the concrete decks and steel beams were also monitored by fixing LVDT at each end of the tested samples, as shown in Fig.2.3. Two load cells were used to record the PT force in the steel tendons during fatigue tests (see Fig.2.3). The data from all the instrumented devices were collected using a data acquisition system, which was controlled by a LabView program. The data was continuously scanned and was stored in a data file every 500 cycles at a rate of 30 samples per second.

2.5 Procedures

The fatigue loads were designed according to the current AASHTO design S-N curve for fatigue capacity of the headed studs shear connectors, which was predicted to fail at one million cycles as shown in Fig.2.4. All samples were tested under the effect of a maximum fatigue load of 160.1 kN and a minimum fatigue load of 13.3 kN, which were 40% and 3% of the maximum static capacity, respectively. The maximum shear stress range in the shear connectors due to a fatigue load range of 146.8 kN was calculated based on the shear flow and transformed section method to be 77 Mpa (El-Zohairy and Salim 2018). Table 2.2 summarizes the five tested samples and Fig.2.5 presents the procedures used to test each
sample. The reference sample (F) without PT was tested under fatigue loading to one million cycles. Sample PF was post-tensioned first, then underwent one million fatigue cycles. Sample EPF was exposed to outdoor environmental, temperature and humidity, changes for 365 days as shown in Fig.2.6. Although exposure to outdoor environment for 365 days is short compared to the service life of a bridge member, it provided valuable insight into the post-tensioning effect on the response of the sample under fatigue cycles. As seen in Fig.2.6, the sample was exposed to short-term and long-term temperature and humidity variations. Since the sample was left outside of the laboratory, only the initiation and propagation of cracks on the slab were visually monitored each month. These cracks were caused by thermal expansion and contraction, which produced flexural deformations and stresses. After the 365 days, the final crack pattern on the concrete deck was recorded. The consequences of these cracks on the fatigue performance will be investigated later in this study. After exposure to outdoor environment for one year, sample EPF was externally post-tensioned and subjected to one million fatigue cycles. Sample FPF was fatigued under one million cycles without PT, then post-tensioned with external tendons and underwent another one million fatigue cycles. Note that sample FPF is the same as sample F but after post-tensioning. That is, after testing sample F under the first one million cycles without PT, the same sample was post-tensioned and tested again, which was labeled as FPF. Sample SPF was loaded statically beyond the yielding strength of the steel beam (85% of its ultimate strength) to cause plastic deformation and then was unloaded resulting in a permanent deflection of 58 mm at the midspan. Finally, the pre-damaged sample (SPF) was strengthened with externally post-tensioned tendons and tested under one million fatigue cycles.
2.6 Results and discussions

The behavior of the tested samples was investigated by monitoring the cracks’ initiation and distribution on the concrete flanges, cyclic incremental deformations (deflections and slippages), cyclic incremental strains in different components of the composite sections, and cyclic incremental PT forces. The maximum envelopes of the data recorded during the fatigue cycles, which correspond to the maximum fatigue load, were used in the curves below to compare the response of each beam during the fatigue cycles. For comparison purposes, all curves were adjusted to start from zero such that the comparison is specific to the fatigue stage of loading and excludes any residual results that were due to the pre-fatigue condition of the samples.

Table 2.2 Test matrix for the tested samples

<table>
<thead>
<tr>
<th>Sample label</th>
<th>Pre-Fatigue condition</th>
<th>Prestressing condition during fatigue testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>Intact</td>
<td>without PT</td>
</tr>
<tr>
<td>PF</td>
<td>Intact</td>
<td>with PT</td>
</tr>
<tr>
<td>EPF</td>
<td>Exposed to outdoor environmental changes</td>
<td>with PT</td>
</tr>
<tr>
<td>FPF</td>
<td>Fatigued to one Million cycles</td>
<td>with PT</td>
</tr>
<tr>
<td>SPF</td>
<td>Permanent deflection of 58 mm</td>
<td>with PT</td>
</tr>
</tbody>
</table>

2.6.1 Fatigue cracks in the concrete slab

During the fatigue testing, cracks on the top and sides of the concrete slab initiated and propagated, whose patterns were monitored as the number of fatigue cycles increased. All observed cracks for all samples were hairline in size as shown in Fig.2.7. Fig.2.8 shows the patterns of the cracks during fatigue for the non-damaged samples. The numbers next to the cracks in the figures indicate the number of cycles at which each crack was initiated.
Sample F had only transverse cracks, most of which initiated around 400,000 cycles. Sample PF, on the other hand, had transverse and longitudinal cracks which extended towards the mid-width and mid-span of the sample. Most of the transverse cracks in sample PF initiated within the first 200,000 cycles, while most of the longitudinal cracks initiated after 400,000 cycles. Both transverse and longitudinal cracks continued to propagate.

![Fig. 2.3 Instrumentation devices.](image)
throughout the rest of the fatigue test. Also, cracks at the sides of the slab of sample PF started to appear after 100,000 fatigue cycles. When comparing the cracks of both samples, sample PF had more and earlier cracks, which indicate a faster rate of degradation in the concrete of the strengthened sample compared to that of the non-strengthened sample. This could be due to the post-tensioning causing minor cracks in the concrete around the shear connectors that extended to the top surface of the slab as the fatigue cycles increased.

For the pre-damaged samples, in addition to the cracks during the fatigue cycles, the pre-existing cracks due to the pre-damages were also recorded and are shown in Fig. 2.9. The cracks that were due to the pre-fatigue conditions are shown in red, while the fatigue crack patterns are represented by blue lines. Sample EPF had cracks in the concrete flange that initiated due to the exposure to the temperature and humidity changes for 365 days, which propagated with additional new cracks during the fatigue cycles as shown in Fig. 2.9(a).

The pre-fatigue cracks in sample EPF were caused by the thermal expansions and contractions that were restrained by the shear connectors. Propagation of these cracks and initiation of new cracks started in sample EPF within the first 200,000 cycles. Also, sample

**Fig. 2.4** AASHTO design S-N curve for fatigue capacity of the headed shear studs.
EPF experienced more total number of cracks and more cracks in the moment span of the beam than all the other samples.

**Fig. 2.5** Testing procedures. Stages F: Fatigue, P: Post-tensioning, E: Environmental changes, S: Static deformation
The pre-existing cracks in sample FPF were due to the fatigue testing of sample F. Although the rate of initiation and propagation of cracks in sample F was slow, initiation of additional transverse cracks in sample FPF appeared after only the first 100,000 cycles and had faster extension rates towards the mid-span and mid-width. This could be due to the degradation of concrete caused by the first one million fatigue cycles combined with the effects of the minor cracks caused by post-tensioning as discussed earlier. Like sample

![Outdoor environmental changes](image-url)

**Fig. 2.6** Outdoor environmental changes.
PF, most longitudinal cracks in sample FPF initiated after 400,000 cycles and were in the shear spans of the samples.

Prior to the fatigue cycles, sample SPF had minor traces of cracks in the sides of the concrete flange. During the fatigue cycles, transverse cracks and minor longitudinal cracks were initiated mostly around 500,000 cycles. This indicates a relatively higher capacity considering that cracks in all the other strengthened samples initiated within the first 200,000 cycles. Also, sample SPF had the least total number and length of cracks compared to the other strengthened samples, including the non-damaged sample (PF). This could be due to the increased strength of the sample caused by plastic deformation.

**Fig. 2.7** Typical hairline crack in the concrete slab (a) side (b) top surface
2.6.2 Cyclic incremental strains in the shear connectors

Figure 2.10 shows the incremental tensile strains in the stud shear connectors located at the ends of the tested samples. As seen in the figure, the external PT reduced the strains in the shear connectors for the strengthened samples compared to the non-strengthened sample (F) and the extent of strain reduction depended on the condition of the sample before strengthening. The strengthened sample with no prior damage (PF) showed the best performance by having the least overall increase in stud strains during fatigue (28 microstrains). Sample SPF exhibited a similar behavior and an overall increase almost as low as the non-damaged strengthened sample PF. This could be due to the plastic

Fig. 2.8 Crack patterns of non-damaged samples. (Numbers of cycles shown next to cracks are in thousands and indicate the initiation of cracks)
deformation increasing the fatigue strength of sample SPF by introducing residual stresses in the steel beam after the load was removed. The effect of the increased strength of sample SPF was also evident by having the least cracks in the concrete flange, which could have also contributed to the relatively improved stud strains behavior of the sample. Sample F had the highest rate of increase and the highest overall increase in stud strains with a total increase of 115 microstrains over the entire fatigue cycles. Sample EPF had a rate of change of the incremental strains like that of sample F until 50% of the fatigue cycles, after which it became almost steady for the remaining fatigue cycles. For samples PF, FPF, and SPF, the rate of change of strains was relatively low and almost constant during the entire test period. The reduction in the cyclic incremental strains in the shear connectors for samples EPF, FPF, SPF, and PF were 25%, 55%, 63%, and 76%, respectively, relative to the reference sample F. This indicates that although post-tensioning caused more cracks on the concrete flange of the strengthened samples, it reduced the imposed stud strains during the fatigue cycles compared to the non-strengthened sample F. After the completion of the testing program, studs near the ends of each of the samples were inspected by removing the surrounding concrete. Although the shear connectors were expected to fail after one million cycles based on the stud fatigue life equations in AASHTO, all the samples survived the applied load level and no failure in the studs was observed. In fact, sample FPF underwent twice the design number of cycles without failure in the studs. This indicates an underestimation of the stud fatigue strength in AASHTO fatigue life equations. The extent of the conservativeness of AASHTO equations could be investigated by performing tests that continue until fatigue failure occurs, which was not done on the samples presented here due to time constraints.
Fig. 2.9 Crack patterns of pre-damaged samples. (Numbers of cycles shown next to cracks are in thousands and indicate the initiation of cracks)
2.6.3 Cyclic incremental deformations

The results of the midspan incremental deflections and the interface slippage at the beam ends are discussed in this section. The deviations in the midspan incremental deflections during fatigue loading are shown in Fig.2.11. Based on the results, the PT effectively reduced the overall increase in incremental deflections of the strengthened samples relative to the non-strengthened sample F. All samples exhibited a significant reduction in deflections except for EPF, whose overall deflection was only 2% less than that of the non-strengthened sample. This could be attributed to the fact that sample EPF had the highest total number of cracks, most of which initiated after 400,000 cycles, which contributed to the degradation of the concrete slab and the overall strength of the sample. The overall increase in deflection for sample FPF was reduced by 45%, while samples PF and SPF were both reduced by 73% relative to that of the non-strengthened sample F. The rate of increase in deflection for sample PF was higher than all the other samples for the first 200 thousand cycles, after which the rate of change became steady and became the lowest. This could be attributed to the relatively faster rate of new crack formation in the sample during the first 200,000 cycles. The overall increase in deflection for samples PF and SPF was also the lowest after the completion of the one million fatigue cycles, which indicates an improved deflection behavior during fatigue for sample SPF caused by its plastic deformation. The incremental interface slippage between the concrete flanges and steel beams at the beam ends is shown in Fig.2.12. The overall incremental slippage for sample F at the end of the fatigue test was 0.016 mm. This level of the incremental slippage was reduced by 11%, 32%, 49%, and 57% for samples SPF, PF, EPF, and FPF, respectively. Samples FPF and EPF had relatively better slippage performance compared to samples PF.
and SPF despite that their pre-damages caused more damages in concrete around shear connectors and led to more incremental deflections during fatigue. This indicates that the overall fatigue behavior was not significantly affected by the partial loss of composite action of the samples.

![Diagram showing incremental strains](image)

**Fig. 2.10** Incremental strains in the shear connectors near supports.

### 2.6.4 Cyclic incremental strains in the extreme fibers

Compressive incremental strains at the top surface of the concrete flanges at midspans are shown in Fig.2.13. The application of PT reduced the incremental increase in the compressive strains for all strengthened samples. The strengthened non-damaged and pre-damaged samples had less incremental compressive strains during the fatigue cycles compared to the non-strengthened sample F. The plastic pre-deformation followed by post-tensioning caused a reduction of 70% in the concrete flange incremental strains in sample SPF relative to the reference sample F. The reductions in samples PF, FPF, and EPF were 55%, 47%, and 38%, respectively, relative to the non-strengthened sample.
Figure 2.14 shows the incremental tensile strains in the bottom flange of the steel beams at midspans during the fatigue loading. The total incremental strain for the reference sample F was 285 micro-strain. The plastically pre-deformed sample SPF experienced the lowest total increase in incremental strains in the steel bottom flange, which was 56% less than that of the reference sample. This could be due to the combined effects of post-tensioning and the residual compressive stresses in the tension flange of the steel beam that were produced after the static overload was removed. In sample PF, the tensile strains increased rapidly during the first 500 thousand cycles. After that, it continued to increase with a lower rate of change until the completion of the fatigue test, which resulted in an overall reduction of 44% in the tensile strains of sample PF due to the application of the PT. The same behavior with a total reduction of 41% was also experienced by sample FPF. The incremental tensile strains in sample EPF were reduced by 24% relative to the non-strengthened sample F.

### 2.6.5 Cyclic incremental PT forces

The incremental deformations due to fatigue cycles led to gradual increases in the post-tensioning force. The incremental increase in the PT force as the number of fatigue cycles increased are shown in Fig.2.15. PT tendons in samples PF and FPF experienced the highest incremental increase in PT forces of 11 kN. The rate of increase was constant throughout the entire fatigue cycles. However, the maximum overall increase represents 13% of the initial PT force and only 6% of the ultimate strength of the tendon, which was not high enough to cause premature failures in the post-tensioned tendons during fatigue testing. Also, the difference in the increase of the PT force between the maximum and
minimum fatigue loads was constant during the entire fatigue tests. These results confirmed that no losses occurred in the PT force due to the fatigue cycles. Similar conclusions were also reached by El-Zohairy et al. [12].

![Graph](image1.png)

**Fig. 2.121** Midspan incremental

![Graph](image2.png)

**Fig. 2.12** Incremental interface slippage

### 2.6.6 Residual capacity of the fatigued beams

After the completion of the fatigue tests, samples PF, EPF, FPF, and SPF were subjected to three-point bending static loading until failure. The static tests were performed to evaluate the residual capacities of the pre-damaged samples with respect to the non-damaged sample PF. The maximum load and maximum deflection of each of the samples are presented in Table 2.3. Compared to sample PF, the maximum load for sample SPF was increased by 7% while experiencing a 40% reduction in the maximum deflection. Samples FPF and EPF had 19% and 13% reductions in the maximum deflection values relative to sample PF with no significant changes in the maximum load values. As presented in Table 2.3, the pre-damages significantly reduced the ductility of the tested
beams while their effects on the maximum loads were insignificant. The effects of these pre-damages on the residual deformations, strains in the shear studs, steel beams, and concrete decks, and failure modes are discussed in detail in (Alsharari et al. 2021).

### Table 2.3 Comparison of the residual maximum loads and deflections

<table>
<thead>
<tr>
<th>Sample</th>
<th>Maximum load</th>
<th>Maximum deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value (kN)</td>
<td>Difference (%) w.r.t PF Value (mm)</td>
</tr>
<tr>
<td>PF</td>
<td>350</td>
<td>0</td>
</tr>
<tr>
<td>SPF</td>
<td>375</td>
<td>7</td>
</tr>
<tr>
<td>FPF</td>
<td>347</td>
<td>-0.85</td>
</tr>
<tr>
<td>EPF</td>
<td>350</td>
<td>0</td>
</tr>
</tbody>
</table>

### 2.7 Summary and Conclusion

In this paper, five steel-concrete composite samples were tested experimentally under fatigue loading. The test matrix was designed to investigate the effect of external PT on the cyclic crack patterns on the concrete flanges, cyclic incremental deformations, and cyclic incremental strains of the tested samples with various pre-fatigue damages. A strengthened sample and a non-strengthened sample were fatigued without pre-damages and were used as references. The remaining samples were strengthened after they were exposed to various damages. The damages were exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damages. The results of this experimental study are summarized below:

- The external PT decreased the strains in the concrete flanges, steel beams, and shear connectors during fatigue loading. However, longitudinal and transverse cracks formed
and propagated in the concrete flanges of the strengthened samples between rows of the studs.

Fig. 2.143 Compressive strains in the deck. Fig. 2.154 Tensile strains in the steel flange.

Fig. 2.15 Incremental PT force during the fatigue tests for the strengthened samples.
• The pre-damage conditions of SCC beams before the strengthening stage affected their responses in terms of strains and deformations during fatigue.

• The non-damaged strengthened sample and the plastically deformed strengthened sample exhibited the most improved performances during fatigue.

• The pre-damage due to fatigue loading or environmental changes caused higher damages in concrete around the shear connectors relative to the plastically pre-deformed strengthened sample and led to more incremental deformations and strains.

• The cracks due to temperature and humidity changes that were formed in concrete around the headed stud shear connectors caused more deterioration in the performance of SCC beams compared to the effect of pre-fatigue.

• The incremental deformations due to fatigue cycles led to a gradual increase in the PT forces, which were not high enough to cause premature failures in the steel strands during fatigue testing.
2.8 References


El-Zohairy, Ayman, Hani Salim, and Aaron Saucier. 2019. “Steel–Concrete Composite Beams Strengthened with Externally Post-Tensioned Tendons under Fatigue.”


Chapter 3: Pre-Damage Effect on the Residual Behavior of Externally Post-tensioned Fatigued Steel-Concrete Composite Beams

3.1 Abstract

Steel-concrete composite beams in highway bridges are susceptible to various types of damage during their service life. These damages can greatly affect the structural performance and shorten the fatigue life of bridges. The efficacy of adding external post-tensioning, as a strengthening technique, to steel-concrete composite beams that are subjected to fatigue loading is not well investigated. Therefore, this study includes fatigue testing of post-tensioned steel-concrete composite beams with various types of pre-damage. Three of the tested samples were subjected to outdoor environmental changes, cyclic preloading, and static overloading as pre-damages before applying the external post-tensioning. The strengthened samples were exposed to fatigue tests to a million cycles under four-point bending. Samples without pre-damage were tested as references to those with pre-damages. The crack patterns in the concrete decks were evaluated during these tests. Static tests to failure were performed to investigate the residual capacities, deformations, and strains of all fatigued samples. The crack patterns in concrete decks were significantly affected by the type of pre-damage that was applied before the post-tensioning. The static overloading pre-damage reduced the number of cracks and their rate of propagation while the exposure to outdoor environment pre-damage induced more
cracks, which negatively affected the crack patterns during fatigue loading. Subjecting the sample to plastic deformation pre-damage slightly improved its performance in terms of residual stiffness and ultimate load. The residual ultimate load was increased by 7% relative to the fatigued sample without pre-damage. However, the ductility was reduced by 40% due to the initial plastic deformation. This reduction in ductility was combined with a decrease in the interface slippage between the concrete deck and steel flange.

3.2 Introduction

Steel-concrete composite (SCC) beams are practical and economical choices for highway bridge designs. The exceptional tensile strength of steel is combined with the compressive strength of concrete to produce sections that are stronger and more cost-effective compared to using either material alone. The steel beam is connected to the concrete deck via shear connectors to transfer the shear and prevent slippage between the top of the steel beam and the bottom of the concrete deck. The shear connectors play a vital role in increasing the static and fatigue strength of the SCC girders (Lorenc and Kubica 2006; Slutter 1966). The static and fatigue strengths are ensured when full composite action is created by ensuring that sufficient shear connectors are provided (Ovuoba and Prinz 2016). However, repeated loads of highway traffic on SCC members often cause gradual strength degradation of the different components of the composite section (Murgudkar and Joshi 2017). A high number of these repeated fatigue cycles usually result in fatigue cracks in the different components of the SCC girders such as the concrete deck, steel beam, and shear connectors. Damages caused by fatigue or other deterioration mechanisms can be avoided, minimized, or repaired by using external post-tensioning. This repairing or strengthening method has
been used for decades by bridge engineers for new and in-service bridges (Klaiber et al. 1988).

Several researchers have investigated the efficacy of post-tensioning under fatigue loading conditions. Albrecht et al. (1995) experimentally determined the fatigue strength of different components of externally strengthened SCC beams. In their study, the focus was on the fatigue strength of shear studs, prestressing strands, and cover plates with low fatigue strength that were welded to the tension flange. Prestressing the beams increased the fatigue strength of the cover plates, which contributes to the overall fatigue strength of the beams. Also, the fatigue strength was affected by the stress range level. The increase in fatigue strength decreased as the stress range level increased. Albrecht and Lenwari (2008) have investigated the effectiveness of several repairing methods in enhancing the fatigue strength and extending the fatigue life of cracked steel beams in steel-concrete composite girders. The composite girders were externally prestressed using high-strength steel tendons. Among the explored repair methods was increasing the prestressing force of already prestressed steel-concrete composite girders. This repair method was very effective and significantly increased the fatigue strength of the cracked samples. They concluded that prestressing both cracked and intact composite girders can make them highly resistant to fatigue damage caused by highway traffic.

The previous studies are relevant examples of the numerous works conducted over the past decades to investigate the effectiveness of externally strengthening with post-tensioned steel tendons under fatigue. However, these studies focused on non-damaged beams and did not address the existence of possible pre-damage scenarios despite that bridge components of existing structures in practice are generally subject to various types of
damages that cause the need for strengthening. Besides fatigue damage, residual stresses due to thermal expansions and contractions can lead to in-service cracks in the concrete deck (Giussani 2009). Also, static overloading conditions cause plastic deformation and yielding of the steel beam of SCC girders, which can affect their structural and durability performances. No previous study has yet addressed the effects of static overloading and environmental effects on the behavior of externally post-tensioned SCC beams. However, the effects of these damages on other types of bridge members are well investigated in the literature. One of the most recent studies was conducted by Xue et al. (2020) on overloaded concrete girders externally strengthened with post-tensioned steel tendons. The study evaluated the effect of initial damage on the performance of post-tensioned concrete beams. Strengthened and non-strengthened concrete beams were tested under static loading conditions after being exposed to different initial damage conditions, namely in-service damage, static overload damage, and undamaged conditions. The in-service and overload damages were simulated by static preloading of the samples with 35% and 85% of the ultimate capacity of the reference samples, respectively. The results show that the initial damage had a slight effect on the flexural ultimate capacity and ductility and a significant effect on the cracking and deflection behavior of the strengthened beams. The study also concluded that post-tensioning was very effective in enhancing the yield strength, ultimate capacity, and flexural stiffness of the damaged beams compared to the non-strengthened beams. Although strengthened concrete girders and SCC girders may have different responses, the study by Xue et al. (2020) gives a general idea of the effect of pre-damage on externally post-tensioned bridge members.
The residual static capacity of fatigued beams can be used to evaluate the effects of various factors on the behavior of SCC beams. The efficacy of external post-tensioning of non-damaged fatigued SCC beams was studied by El-Zohairy et al. (2019b). The degradation of the different components of the steel-concrete composite section was studied by evaluating the residual flexural capacities of strengthened and non-strengthened fatigued beams as well as the static capacity of non-fatigued beams. The residual static capacities of the strengthened and non-strengthened fatigued beams were compared after being subjected to the same number of fatigue cycles. The results indicate that the prefatigued samples experienced a reduction in ultimate load capacity as well as ductility compared to the unfatigued sample. The prefatigued samples also experienced increased residual strains in the steel beams and concrete flanges, increased midspan deflections, and increased interface slippages. The study also showed that the strengthened prefatigued sample had better residual capacity compared to the non-strengthened prefatigued sample.

After reviewing the current literature, no studies were found that address the residual behavior of damaged prefatigued SCC beams that were externally strengthened with post-tensioning tendons. Existing literature related to the strengthening of existing bridges focused on reinforced concrete bridge girders or SCC beams that are strengthened with methods other than steel tendons such as carbon fiber reinforced polymers. Also, the current studies mostly focused on the monotonic behavior of bridge members without considering the effect of undergoing fatigue cycles. Furthermore, the effects of possible pre-damages on the efficacy of strengthening of SCC beams were not studied. Thus, the present study is an attempt to expand the state of knowledge by evaluating the effects of the pre-existence of damage on the performance of strengthened SCC beams. The
evaluation was carried out experimentally by comparing the residual performance of damaged and non-damaged fatigued steel-concrete composite beams strengthened with externally post-tensioned steel tendons.

3.3 Experimental program

3.3.1 Test samples

Five steel-concrete composite beams were tested in this study. All samples were identically designed and fabricated. All samples were conditioned one year before the testing program started; four of which were conditioned indoor, and one sample was conditioned outdoor. The concrete in all samples was cast and cured similarly. The beams were 4752 mm in length. The dimensions and reinforcement details of the tested beams are shown in Fig.3.1. The full composite action was ensured by welding two rows of stud shear connectors spaced at 122 mm apart in the longitudinal direction according to ASSHTO LRFD Bridge Design Specifications (AASHTO 2018). The stud shear connectors were 15.875 mm in diameter and 80 mm in height. Stiffeners at supports and loading points were welded to the steel beams to avoid the local buckling of the steel webs. Seven-wire high strength steel strands with a yield strength of 1680 Mpa were located 32 mm above the bottom flange of the steel beam for the external post-tensioning. Head plates were used to anchor the post-tensioning tendons to the ends of the beams. The steel beams were grade A992 with a yield strength of 345 Mpa. The yield strength of the shear connectors was 352 Mpa. The average concrete compressive strength was 33 Mpa. Pre-damages were induced in three samples before being strengthened with the external post-tensioning. The applied pre-damages in this study were exposure to outdoor environmental changes for 365 days, cyclic preloading
to simulate the in-service fatigue damages, and static overloading beyond the yield strength to induce permanent deflections. The remaining two samples were strengthened and used as control samples without any prior damage. The details of the test matrix are summarized in Table 3.1. In this paper, the samples are described with acronyms that indicate their loading histories as detailed in Table 3.2. Accordingly, sample PS was post-tensioned first and then the static test was applied to failure. This sample was used as a control beam without fatigue loading to evaluate the residual capacity of the fatigued sample without pre-damages (PFS). Sample PFS was post-tensioned and then subjected to fatigue loading of one million cycles followed by a residual static test to failure. This sample was used as a control sample for the other samples with pre-damages. Sample EPFS was left outside the testing laboratory for one year to be exposed to environmental changes such as temperature and humidity. The thermal expansions and contractions due to the daily and seasonal environmental changes caused visible cracks on the concrete surface and damage in concrete around the shear connectors. The sample was then externally post-tensioned and tested under fatigue loading of one million cycles followed by a static test to failure. Sample FPFS was pre-damaged by undergoing one million fatigue cycles without post-tensioning to simulate the actual condition of non-strengthened bridge beams under the effect of traffic loading. The post-tensioning was applied after the completion of cyclic preloading. The sample was then loaded under another one million fatigue cycles followed by a static test to failure. The last sample OPFS was subjected to static overloading beyond its yielding strength to induce plastic deformations and then was unloaded. Later, the external post-tensioning was applied, and then the sample was tested to one million fatigue cycles followed by static loading to failure.
3.3.2 Instrumentation

The midspan deflection, steel-concrete interface slippage, midspan strains in the extreme fibers of the composite section, and strains in the shear connectors near supports were monitored in each sample. Fig.3.2(a) shows the types and locations of the monitoring devices used in this study. Linear pattern precision pre-wired strain gauges (SG) were attached to the shear connectors in the regions of high shear values according to the shear diagram. These strain gages were attached to both sides of the instrumented connectors to monitor the tensile and compressive strains of each side. Surface-mounted strain transducers were placed on the concrete decks at midspans and near midspans to monitor the compressive strains (Fig.3.2 (b)). The bottom surfaces of the steel beams were instrumented at midspans with SG to record the tensile strains.

String potentiometers and Linear Variable Differential Transducers (LVDTs) were used to record the midspan deflections and horizontal LVDTs were used to record the interface slippages at the ends of the samples. The forces in the post-tensioned tendons were monitored using two load cells located at the dead-end of the tendons (Fig.3.2 c).

3.3.3 Loading setup and procedure

The testing program was performed in four stages. The first stage was inducing the pre-damages in the three selected beams as described previously and illustrated in Fig.3.3. The daily and seasonal variations in temperature and humidity subjected to sample EPFS are shown in Fig.3.3(a). These environmental variations led to thermal cracks that were monitored and will be discussed later in this paper. Sample OPFS was loaded as shown in
Fig. 3.3(b). Considering that 40% of bridges in the United States are 50 years or older with 9% classified as structurally deficient according to (ASCE 2017), significant damages can occur when deteriorated aging bridges are subjected to possible overload scenarios. Thus, sample OPFS was intentionally overloaded beyond its typical in-service load to cause

![Diagram of OPFS dimensions and reinforcement details](image)

**Fig. 3.1** Dimensions and reinforcement details for (a) Cross-section and (b) Elevation. (Dimensions shown are in mm)
Table 3.1 Summary of the loading scheme

<table>
<thead>
<tr>
<th>Sample*</th>
<th>Pre-damage type</th>
<th>Fatigue testing after post-tensioning</th>
<th>Static loading after completion of fatigue tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS</td>
<td>None</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>PFS</td>
<td>None</td>
<td>One million cycles</td>
<td>Static loading to failure</td>
</tr>
<tr>
<td>EPFS</td>
<td>Exposure to environmental changes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FPFS</td>
<td>Fatigue loading to one million cycles</td>
<td>One million cycles</td>
<td></td>
</tr>
<tr>
<td>OPFS</td>
<td>Static overloading beyond the yield strength</td>
<td>Static loading to failure</td>
<td></td>
</tr>
</tbody>
</table>

* Naming convention is described in Table 3.2.

Table 3.2 details of the naming convention system

<table>
<thead>
<tr>
<th>Letter</th>
<th>Its indication</th>
</tr>
</thead>
<tbody>
<tr>
<td>P</td>
<td>Post-tensioning</td>
</tr>
<tr>
<td>F</td>
<td>Fatigue loading to one million cycles</td>
</tr>
<tr>
<td>E</td>
<td>Exposure to outdoor environmental changes</td>
</tr>
<tr>
<td>O</td>
<td>Overloading beyond the yielding strength (static)</td>
</tr>
<tr>
<td>S</td>
<td>Static loading to failure</td>
</tr>
</tbody>
</table>

permanent damage and subsequently study the post-tensioning effect. The overload was reached experimentally by statically loading the sample while observing the strain measurements in the tension flange of the steel beam. The loading was continued until plastic strains were observed to ensure plastic deformations and plastic damage in the steel beam were recorded. Plastic deflection of the steel beam was observed at a load of 350 kN, which is approximately 85% of the expected static capacity of a non-strengthened SCC beam. After unloading, the resulting permanent midspan deflection was measured to be 58 mm.
Sample FPFS was subjected to fatigue loading with a maximum of 45% of the sample yielding load (within the elastic range). The fatigue load range was based on a desired number of fatigue cycles of one million cycles, which corresponds to a stress range of 77.4 MPa based on AASHTO’s lower bound S-N curve equations for the headed stud shear connectors (AASHTO 2018). A lower stress range of 50 MPa, for example, which is just above the constant amplitude fatigue limit, requires 500% more time per sample than the stress range used in this study. Thus, the current stress range was selected so that the testing program can be completed within a reasonable time considering the significant time high cycle fatigue tests may take. According to AASHTO’s fatigue capacity equations, this shear stress range was expected to cause the shear connectors to experience initial damage after one million cycles. Values of maximum and minimum shear stress in the shear connector for the cyclic preloading are shown in Fig.3.3©. The shear stresses were calculated based on the shear flow equation and the transformed section properties (El-Zohairy and Salim 2018).

The second stage of the testing program was the application of the external post-tensioning as a strengthening technique for the tested samples. All beams were initially post-tensioned to a force of 85 kN (46% of its ultimate strength). The third stage was subjecting the samples to one million fatigue cycles. The fatigue test of this stage was identical to that of the first stage (see Fig.3.3©). The fatigue load was applied at a frequency of 4 HZ using a single jack hydraulic actuator equipped with a spreader beam to distribute the applied load to two loading points as shown in Fig.3.4(a). The fourth stage was residual static tests to failure under the effect of one concentrated load at the mid-spans of the tested samples (see Fig.3.4(b)).
3.4 Results and discussion

3.4.1 Crack patterns in the concrete decks

After the completion of each testing stage, cracks on the top and side surfaces of the concrete decks were drawn. The cracks that were initiated and propagated due to the pre-damages induced in the tested samples (drawn in red) and during the fatigue loading (drawn in blue) are shown in Fig.3.5. All samples experienced longitudinal and transverse cracks due to fatigue loading. Fig. 3.5(a) illustrates the cracks that initiated and propagated due to fatigue loading on the concrete deck of the reference sample PFS. These cracks could be
attributed to the combined effect of the pre-existing minor cracks that were initiated by the cement hardening and the camber caused by the post-tensioning, which caused additional minor cracks near the stud connectors. These minor cracks were further propagated and caused new surface cracks during the one million fatigue cycles loading stage. Fig. 3.5(b) shows minor side cracks due to the overloading of sample OPFS with no cracks on the top surface of the concrete deck. During fatigue loading, cracks were initiated on the top surface which were propagating at a rate less than that of sample PFS. The strain hardening caused by the overloading and plastic deformation increased the dislocation density and yield strength of the overloaded beam during fatigue loading. This, in turn, enhanced its fatigue-induced damage resistance even when compared to the undamaged sample PFS.

Fig. 3.5(c) shows the crack pattern in the concrete deck of sample FPFS after the two fatigue tests (since the pre-damage was induced by a fatigue test). During the first fatigue test, only transverse cracks (drawn in red) were observed. The longitudinal cracks initiated and propagated only during the second fatigue test (after strengthening) due to the damage near studs caused by the post-tensioning camber. Fig. 3.5(d) shows the thermal cracks (drawn in red) and fatigue cracks (drawn in blue) in the concrete deck of sample EPFS. The thermal cracks were caused by the daily and seasonal changes of temperature and humidity that caused expansion and contraction of the different components of the composite beam. Since movements of both the steel beam and concrete deck were restrained by the presence of shear connectors, these environmental variations resulted in thermal cracks in the concrete deck. The blue lines in Fig. 3.5(d) represent the cracks that initiated and propagated during fatigue loading which were less than those of sample FPFS.
However, the final crack pattern in the concrete deck of sample EPFS was the worst compared to the other pre-damaged samples.

**Fig. 3.3** Different types of the pre-damage applied to the tested samples.

### 3.4.2 Fatigue loading effect on the residual deformation

To evaluate the effect of fatigue loading on the residual performance of the post-tensioned samples, the results of the two samples PS and PFS were compared. The residual
deformations of these two un-damaged samples are shown as black lines in Fig. 3.6. The load-deflection relationships illustrated in Fig. 3.6(a) showed reductions in both the ultimate load-carrying capacity and ductility by 13% and 17%, respectively, due to fatigue loading. Moreover, the stiffness of the fatigued sample was reduced by 10% relative to the un-fatigued sample. The steel beam yielded at 266 kN for the fatigued sample with an 18% reduction in the yield strength of the un-fatigued sample due to fatigue loading.

These reductions were mainly due to the degradation in the shear connection strength between the steel beam and concrete deck caused by concrete damage around the shear connectors combined with the cracks on the concrete deck surfaces due to fatigue loading.
The degradation in the shear connection strength is illustrated in Fig. 3.6(b) by comparing the relative slippage of the two samples. The slippage for sample PS increased linearly until yielding, after which the slippage increased to 0.23 mm at the ultimate capacity. The slippage of sample PFS confirmed the damage in concrete around the shear connectors, which was more pronounced when the applied static load reached the level of the maximum fatigue loading. The total slippage increased by 25% for the fatigued sample relative to the un-fatigued one.

### 3.4.3 Pre-damage effect on the residual deformations

The pre-damage effect on the residual deformations was evaluated relative to the initially un-damaged samples (PS and PFS). The load-midspan deflection relationship for sample OPFS, shown in Fig. 3.6(a), showed that subjecting the sample to plastic deformation slightly improved its performance in terms of residual stiffness and ultimate load. The residual ultimate load was increased by 7% relative to sample PFS. However, the ductility was reduced by 40% due to the initial plastic deformation. This reduction in ductility was accompanied by a reduction in the slippage (see Fig. 3.6(b)).

The effect of the initial fatigue test on the residual deformations of sample FPFS is also evaluated in Fig. 3.6. A reduction of 8% in the ultimate capacity occurred. However, the slippage increased by 20% relative to sample PFS. Undergoing fatigue loading twice (two million cycles; once before post-tensioning and another time after post-tensioning) caused relatively more loss in the composite action between the concrete deck and steel beam, which is reflected in the load-slippage relationships as shown in Fig. 3.6(b). The specimen was inspected for fatigue failure in the shear studs after the completion of the residual static
test. No failures occurred in the shear connectors due to the fatigue loading of two million cycles. It is worth noting that the current AASHTO stud design was reported by Ovuoba and Prinz (2016) and Provines et al. (2019) to significantly underestimate the stud fatigue capacity. The lack of fatigue cracks was also reported on existing SCC girders that were examined after having exceeded their AASHTO’s fatigue life expectancy (Ovuoba 2017). The overestimation of stud fatigue strength was thought to be partially due to friction between the steel beam and the concrete deck that occurs at the steel-concrete interface, which is not accounted for in the current AASHTO design equations (Ovuoba 2017). This friction contributes to the shear stress resistance of the SCC girders, making the shear stress range experienced by the shear studs lower than the design shear stress range. Thus, the lack of fatigue cracks of the inspected studs in sample FPFS could be attributed to the conservative stud fatigue life expected based on AASHTO’s S-N curve for stud connectors, which was used in the current study.

Results of the residual deformations after the exposure to environmental changes for sample EPFS are also shown in Fig 3.6. Although the final crack pattern in the concrete deck for this sample was the worst compared to the other samples with other types of pre-damages, no significant changes in the ultimate load-carrying capacity and slippage were observed. The ductility and yield strength were reduced by 10% and 12%, respectively. Thermal cracks were induced in the concrete deck due to the restrained movement of the steel beam and concrete deck. These cracks did not affect the strength of the composite interaction as the residual slippage values for sample EPFS were like that of the undamaged fatigued sample FPS. Table 3.3 summarizes the variations in the ultimate load capacities and ultimate midspan deflections for the tested samples.
3.4.4 Pre-damage effect on the shear connectors’ strains

Higher incremental strains during fatigue loading were expected in the shear connectors near the supports in comparison to the other connectors according to El-Zohairy et al. (2019a). This difference in the incremental strain correlates to the relative slippage between the concrete deck and steel beam along the beam length, which was maximum at the beam ends. Therefore, strains in the shear connectors near supports were monitored for each sample in this study as shown in Fig. 3.7. The pre-damage effect on the tested samples varied based on the type of pre-damage. Sample OPFS had the lowest strains in the shear connectors while sample FPFS had the highest strain values of the pre-damaged samples. The shear connectors in sample EPFS exhibited the same strain level as in sample PFS, which is also confirmed by the slippage results (see Fig. 3.6(b)). These results confirmed that exposure to outdoor environmental changes for one year had no significant effect on the degree of composite interaction. To consider the effect of fatigue loading only, strains in the shear connectors of the un-damaged samples PFS and PS were compared. The fatigue loading caused damage in concrete around the shear connectors which led to an increase in the slippage of sample PFS (see Fig. 3.6(b)) resulting in increased strains in the shear connectors in sample PFS compared to the unfatigued sample PS (see Fig. 3.7).

3.4.5 Pre-damage effect on the extreme fibers’ strains

The tensile and compressive strains in the bottom flange of the steel beams and top surface of the concrete decks, respectively, are compared in Fig. 3.8 to evaluate the effect of the pre-damage on these strains. In general, the fatigue cycles increased the midspan steel beam strains for samples PFS, EPFS, and FPFS compared to sample PS. The pre-damage in
samples EPFS and FPFS did not cause an additional increase in the strains in the steel beams and they behaved like the undamaged sample PFS. However, sample OPFS exhibited an enhanced response via increasing in the steel beam strains due to the reductions in ductility and slippage caused by the plastic deformation. The compressive strains in the concrete decks at midspans are presented in Fig. 3.8(b). Sample FPFS had the same response as sample PFS where both experienced an increase in the compressive strains of approximately 40% relative to sample PS. The strains in sample EPFS increased in a slope similar to that of sample PFS until a load level of 200 kN (60% of the ultimate load). Afterward, the slope increased rapidly until the maximum load was reached resulting in an overall 12% increase in the total strain values. Sample OPFS had the least compressive strains in the concrete among all the fatigued samples.

(a) Cracks at the top and side surfaces of the concrete deck of sample PFS

(b) Cracks at the top and side surfaces of the concrete deck of sample OPFS
3.4.6 Failure modes of the residual static tests

Yielding in the steel beams followed by crushing in the concrete decks was the observed failure mode for all the tested samples under the residual static tests as illustrated in Fig. 3.9. The tests were stopped after the ultimate capacities were reached, at which the applied load started to decrease. After the completion of the residual static tests, the concrete around the shear connectors was removed and the stud shear connectors were visually examined. No failures in the shear connectors were observed, which was verified by the slippage values and shear connectors strains as discussed previously.
Fig. 3.6 Pre-damage effect on (a) Load-deflection relationships, and (b) Slippage
Table 3.3 Variations in the ultimate load capacities and midspan deflections.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate load capacity</th>
<th>Ultimate midspan deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value (kN)</td>
<td>Normalized*</td>
</tr>
<tr>
<td>PS</td>
<td>400</td>
<td>1</td>
</tr>
<tr>
<td>PFS</td>
<td>350</td>
<td>0.875</td>
</tr>
<tr>
<td>OPFS</td>
<td>375</td>
<td>0.938</td>
</tr>
<tr>
<td>FPFS</td>
<td>347</td>
<td>0.868</td>
</tr>
<tr>
<td>EPFS</td>
<td>350</td>
<td>0.875</td>
</tr>
</tbody>
</table>

*Normalization was done with respect to sample PS.

Fig. 3.7 Pre-damage effect on the shear connectors’ strains
Summary and Conclusions

This study includes fatigue testing of post-tensioned steel-concrete composite beams with various types of pre-damage. Three of the tested samples were subjected to various damages: outdoor environmental changes, cyclic preloading, and static overloading, as pre-damages before applying the external post-tensioning. The strengthened samples were exposed to fatigue tests to a million cycles under four-point bending. Two samples without pre-damage were evaluated as references to those with pre-damage. The crack patterns in the concrete decks were evaluated during these tests. Finally, residual static tests to failure were performed to evaluate the residual capacities, deformations, and strains of each sample. Based on the test results of this study, several conclusions can be drawn:

Fig. 3.8 Residual strains in the (a) Bottom flange of the steel beams, and (b) Top surface of the concrete decks.

3.5 Summary and Conclusions
• The crack pattern in concrete decks was significantly affected by the type of the pre-damage, which was applied before the post-tensioning strengthening. The pre-damage in the form of plastic deformation reduced the number of cracks and their rate of propagation, while the exposure to thermal pre-damage induced more cracks, which negatively affected the crack patterns during fatigue loading.

• The fatigued sample without pre-damage experienced reductions in the yield strength, ductility, ultimate load-carrying capacity, and stiffness by 18%, 17%, 13%, and 10%, respectively, relative to the un-fatigued sample without pre-damage.

• Subjecting the sample to static overloading slightly improved its performance in terms of residual stiffness and ultimate load. This improvement in the response can
be attributed to the strain hardening caused by plastic deformation. The residual ultimate load was increased by 7% relative to the fatigued sample without pre-damage. However, the ductility was reduced by 40% due to the initial plastic deformation. This reduction in ductility was combined with a decrease in the steel-concrete interfacial slippage.

- The reductions in ductility and slippage that were caused by the plastic deformation in sample OPFS caused an enhanced response via decreasing the steel beam strains. Moreover, this sample had the least compressive strains among all the fatigued samples.

- Although the final crack pattern in the concrete deck for the sample with the outdoor environmental changes pre-damage was the worst compared to the other samples with different types of pre-damages, no significant changes in the ultimate load-carrying capacity and slippage were observed. The ductility and yield strength were reduced by 10% and 12%, respectively.

- The pre-damage effect on the shear connectors’ strains varied based on the type of damage. Sample OPFS had the lowest strains in the shear connectors while sample FPFS had the highest values.

- The pre-damage type did not change the failure mode after the residual static tests. The typical failure mode observed for all samples was yielding in the steel beam followed by crushing in the concrete deck.
3.6 References


Federal Highway Administration (FHWA), Report No. FHWA-HRT-20-005.


Chapter 4: Numerical Investigation of the Monotonic Behavior of Strengthened Steel-Concrete Composite Girders

4.1 Abstract

The monotonic behavior of externally post-tensioned steel-concrete composite girders was numerically studied in this paper. A three-dimensional numerical model was developed and validated using experimental test results that were conducted by the authors. A parametric study using this validated numerical model was performed to investigate the effects of various parameters on the monotonic performance of composite girders strengthened with external post-tensioned tendons. The parameters investigated include variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the studs. The numerical model provided a better understanding of the effect of these parameters on the behavior of the strengthened beams. The results of the parametric study show that the slippage between the concrete deck and steel beam increased as the degree of shear connection decreased. Also, as the shear connection degree decreases, its effect on the slippage behavior increases. The study also shows that, for the same degree of shear connection, beams with one row of shear studs had higher flexural capacity than beams with two rows of studs. The load-deflection and slippage behavior improved when smaller diameters of the studs were used. The higher the post-tensioning force the higher the ultimate load capacity and the lower the tensile strains in the steel beam. The tensile strains at midspan were
considerably reduced by increasing the depth of the steel beam. The lowest midspan tensile strains were obtained from the combination of increasing the depth of the steel beam and tendon eccentricity.

4.2 Introduction

External post-tensioning of steel-concrete composite (SCC) girders is commonly used in highway bridges as a strengthening method. Typically, the strengthening is done using high-strength steel tendons positioned near the tension fiber of the composite girder. The flexural performance of the SCC girders is largely affected by the performance of its individual components, such as the steel beam, concrete slab, slab reinforcement, and shear connectors. In the literature, the behavior of non-strengthened SCC girders under monotonic loading is well investigated, experimentally and numerically. Alwash and Jaber (2015) studied the effect of the number and distribution of headed stud shear connectors on the load-slip and ultimate load of composite girders. Various spacing distributions and various numbers of shear connectors were considered. They studied the effect of increasing the spacings between studs in the regions near the midspan and decreasing them at the beam ends while maintaining the same number of studs. This spacing distribution caused a reduction in the slippage of the steel-concrete interface, which resulted in an improved flexural capacity of the beams. Also, the study showed that the overall strength of the composite girders improved as the number of shear connectors increased. A 35% increase in the number of shear connectors resulted in a 42% reduction in the midspan deflection of the beam with a 2% increase in the ultimate load value. Wang et al.(2018) experimentally studied the static behavior of four SCC girders with four degrees of shear
connection. As the degree of shear connection decreases, they reported flexural failure modes for beams with shear connection degrees equal to or greater than 1. When the degree of shear connection was around 60%, the failure mode changed to fracture of the studs followed by separation of the concrete deck from the steel beam. Also, the slippage along the length of the SCC beams was reported to increase as the shear connection degree decreases in addition to reductions in the ultimate capacity and ductility coefficient. Liu et al. (2019) investigated the effects of various parameters on the flexural strength of SCC beams under hogging moments. The studied parameters included variations in the degree of shear connection, arrangement and diameter of the shear connectors, the longitudinal and transverse reinforcement ratio, and the length of the sample. The results of their experimental and numerical study suggest that the flexural strength is greatly affected by the degree of shear connection and the ratios of the steel reinforcement, while the impact of the other parameters was insignificant.

Several researchers conducted FE studies to simulate and investigate the flexural behavior of non-strengthened SCC beams. Amar Prakash et al. (2012) developed a 3D finite element model to study the behavior of steel-concrete composite girders connected by shear studs. Their proposed model, validated by experimental results, was able to predict the energy absorption capacity, load-slip at the steel-concrete interface, stud’s shear force capacity, and the failure mode of the girders. Lam and El-Lobody (2005) proposed a numerical model to simulate the shear stud behavior in composite beams and numerically investigated the effect of variations in concrete strength and diameter of shear stud on the shear capacity of shear studs. Zheng et al. (2021) performed a parametric study to study the effects of parameters such as concrete compressive strength and height of the steel beam on the
ultimate bearing capacity and slippage of SCC beams. They reported that as the compressive strength of concrete increases, its effect on the ultimate load and slippage behavior becomes insignificant. They also reported that increasing the steel beam height led to significant improvement in the ultimate load capacity while it has an insignificant effect on the slippage response.

A few studies exist on the flexural behavior of externally post-tensioned SSC beams. Uy and Craine (2004) conducted experimental and numerical tests on post-tensioned SCC beams. They developed a numerical model to evaluate the effect of various parameters on the behavior of strengthened SCC beams. The parameters included the slab reinforcement and dimensions, initial force and eccentricity of the tendon, and concrete strength. The study reported that the strength, stiffness, and ductility of the beams increase as the area of slab reinforcement increases. Also, the ductility of the SCC beams was significantly increased by increasing the width of the slabs. Furthermore, the increase of the initial force of the tendon increased the moment-carrying capacity of the beams with a reduction in ductility. Lorenc and Kubica (2006) studied the effect of various parameters related to the different components of the strengthened SSC beam on the local and global SCC behavior under positive bending. In their study, the effect of shear connection flexibility, external post-tensioning, the profile of the tendons (draped or straight), and concrete compressive strength were explored. The results of the tests indicate that the performance of the beams was improved by adding the external tendons while the profile of the tendons having no significant impact on the ultimate load capacity. Also, the beams with higher concrete strength exhibited an increased load-carrying capacity and a reduced interface slippage compared to the samples with lower concrete strength. A few finite element studies were
conducted to simulate the behavior of strengthened SCC beams. Chen and Jia (2010) developed a FE model to investigate the behavior of SCC beams strengthened with external tendons. The proposed model was used to conduct an inelastic buckling analysis of SCC beams in negative bending. The study reported that the variation in the prestressing force applied to the SCC beam was amongst the most important factors affecting the behavior of strengthened SCC beams. Parametric studies were conducted by El-Zohairy and Salim (2017) and El-Zohairy et al. (2017) to evaluate the performance of strengthened composite beams with different degrees of shear connection, prestressed region length, and tendon eccentricity. Prestressing the entire length of the beams improved the behavior of the beams by reducing cracks that appear due to stress concentrations near the anchorage areas when the tendon length is not extended to the full length of the beams. The stiffness and the capacity of the beams increased with the increase in shear connection degree. Also, the study suggested that increasing the post-tensioning force caused a considerable reduction in the tensile stresses.

Most of the current literature is focused on the flexural behavior of non-strengthened SCC beams, while only a few studies focus on externally strengthened beams. Thus, this study is an attempt to fill gaps in the literature and address the lack of studies on the behavior of strengthened SCC beams. Also, due to cost and time factors, previous experimental and numerical studies usually focus on a few parameters related to the behavior of SCC beams while ignoring many other important parameters. Therefore, the objective of this study is to develop a three-dimensional finite element model that can simulate the behavior of composite steel-concrete girders strengthened with external post-tensioned steel tendons. The model, validated against the results of experimental tests by Alsharari et al. (Alsharari
et al. 2021), will then be used to perform a parametric study to investigate the effects of variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the stud on the flexural behavior of the strengthened composite girders.

4.3 Development of the Finite Element Model

A sophisticated three-dimensional finite element (3D-FE) model of SCC girders requires a proper representation of the geometric and constraint conditions of each individual component of the composite structure. The typical components of a composite girder include the steel beam, concrete deck, shear connectors, longitudinal and transverse reinforcements of the deck, loading and support plates, stiffeners, and the strengthening system. For this investigation, the finite element modeling package Ansys (v. 18.1) was selected. The steel I-beam and steel plates were modeled using SOLID185 elements. This eight-node element was used for the three-dimensional modeling of solid structures (Canonsburg 2011). At each of the nodes, SOLID185 contained three degrees of freedom: translations in the nodal x, y, and z directions. Its capabilities include plasticity, stress stiffening, large deflection, and large strain. The concrete was modeled using SOLID65 elements. The most important feature of this element is its capability of crushing in compression and cracking in tension. It also has three degrees of freedom at each of its eight nodes. The longitudinal and transverse slab reinforcements, as well as the shank of the stud shear connectors, were modeled using BEAM188 elements. Non-linear spring elements, COMBIN39 and COMBIN14, were used to simulate the parallel and
perpendicular movements of the studs relative to the direction of the steel beam. A four-
node shell element, SHELL181, with six degrees of freedom at each node, was selected to
model the steel stiffeners and anchor plates that were welded to the steel beam. The post-
tensioning tendons were modeled using LINK180. The interaction between the top surface
of the steel beam’s top flange and the concrete deck was considered by utilizing a contact
element, CONTAC174, to avoid the possible penetration between the two surfaces.
ANSYS’s CONTA174 and TARGE170 were used to model the contact and target surfaces,
respectively. CONTA174 defined surfaces with deformation capabilities and simulated
their interaction with three-dimensional surfaces that were specified as targets. It shared
the geometric features with the face of the solid element to which it was connected.
TARGE170 defined various three-dimensional target surfaces, divided into segments and
associated with the corresponding contact elements. In this paper, the shear connectors
were selected as contacts, and the concrete flange was selected as a target body.

The external post-tensioning force F was applied as an equivalent negative temperature
difference based on the following equation:

\[ F = EA \alpha \Delta T \]  

(1)

Where E is Young’s modulus, A is the area of the tendons, \( \alpha \) is the coefficient of thermal
expansion (\( 10e^{-5} \) for steel), and \( \Delta T \) is the temperature difference. The meshing, loading
setup, and details of the different components of the composite beam are presented in Fig.
4.1. A convergence assessment shows that 45,072 elements and 65,520 nodes are suitable
for calculating the stresses and deformations. The boundary conditions of the three-point
bending tests were modeled to represent the experimentally tested simply supported
composite girders. The slippage at the interface between the steel beam and the concrete
flange was taken into account by inputting the load-slip data of headed shear connectors in the real constants of the spring elements. This load-slip relationship, presented in equation 2, was first introduced by Ollgaard et al. (1971) and was since used by other authors such as Johnson and Molenstra (1991).

\[ P = P_u (1 - e^{-\beta \delta})^\gamma \quad (2) \]

where \( P \) is the horizontal shear force in the shear connector, \( P_u \) is the design capacity of the shear connector, \( \delta \) is the steel and concrete interface slippage and \( \beta \) and \( \gamma \) are coefficients that control the stiffness of the studs. In this study, different values of \( \beta \) and \( \alpha \) were examined, and the values that gave the closest numerical results to the experimental results were 0.6 and 1, respectively. The uniaxial stress-strain relationship for concrete in compression is obtained from the following equations to represent the multi-linear isotropic stress-strain curve for the concrete (Desayi and Krishnan 1964).

\[ f = \varepsilon E_c \quad \text{for} \quad 0 \leq \varepsilon \leq \varepsilon_1 \quad (3) \]

\[ f = \frac{\varepsilon E_c}{1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^2} \quad \text{for} \quad \varepsilon_1 \leq \varepsilon \leq \varepsilon_0 \quad (4) \]

\[ \varepsilon_0 = \frac{2f_c}{E_c} \quad (5) \]

Where \( f \) is the compressive stress at any strain \( \varepsilon \), \( E_c \) is concrete Young’s modulus, \( \varepsilon_1 \) is the strain corresponding to \( 0.3f_c \), \( \varepsilon_0 \) is the strain at the ultimate compressive strength \( f_c \). The behavior of concrete in tension is assumed linear up to the tensile strength, after which the tensile resistance reduces progressively to zero due to the cracking of concrete. A bilinear stress-strain curve was used to represent the stress-strain relationships of the steel.
components of the composite girders. The material properties used in both the experimental
tests and the finite element model are summarized in Table 4.1.

![Fig. 4.1. Finite element model; loading setup (top), meshing (bottom left), and details of the different components of the composite beam (bottom right).]

4.4 Validation of the 3D-FE Model

The proposed 3D-FE model was validated using the results of experimental tests performed by Alsharari et al. (2021). The experimental testing was conducted on strengthened composite steel-concrete girders with a support span of 4.42 meters. The strengthening was done using externally post-tensioned straight steel tendons positioned 32 mm above the bottom flange of the steel beam. Two rows of headed stud shear connectors were used to attain full shear connection per AASHTO LRDF Bridge Design Specifications (AASHTO
Fig. 4.2 shows the dimensions of the cross-section geometry, and Fig. 4.3 shows the details of the different components of the sample in the direction parallel to the beam span. The experimental and the FE tests were displacement-controlled with the loading applied to the midspan of the beams using a steel plate to avoid stress concentration.

Results obtained from the experimental program and the finite element model were compared in Fig. 4.4 and Fig. 4.5 to evaluate the accuracy of the model to simulate the behavior of the strengthened sample. The monotonic behavior of the girders was compared in terms of load-deflection curves, strains in the concrete deck, strains in the stud connectors, and failure modes. Fig. 4.4a shows that the difference in the ultimate loads of
the numerical and experimental tests was within 1%. Also, the yield strength and ultimate
deflection obtained from the model agreed well with that of the experimental results. As
can be seen in the figure, the experimentally tested beam reached the maximum load then
started to decrease with a rate higher than that of the FE model. This could be caused by
the assumption of full bonding between the concrete and steel reinforcement of the slab,
by the force-slippage equation that was used for the shear connectors, and/or by the
concrete constitutive model. In this study, special attention was paid to the behavior of the
composite beam before the maximum load was reached. Therefore, the solution was
stopped when the displacement of the loading plate reached the maximum value, which
was pre-defined beyond the experimental displacement corresponding to the maximum
load. Strains in the stud connectors near the ends of the composite girders are shown in
Fig. 4.4b. The results obtained from the model were within 1% of the experimental results.
Fig. 4.4c shows the increase in the concrete deck strains at midspan as the load increases.
The model had less concrete strain values at all load levels, with a maximum difference of
30%. This difference could be attributed to the way the experimental concrete strains were
measured using surface-mounted strain transducers.

Gage length extension, which improves the readings in non-homogeneous materials such
as concrete, was not used in the experimentally tested beam. Fig. 4.5 shows the comparison
between the experimental and FE model modes of failure. The failure modes of both the
experimental and FE model were concurrent yielding of the bottom flange of the steel beam
at midspan and crushing of the concrete deck at midspan.
Fig. 4.4. Comparison between numerical and experimental results; a) Load-deflection, b) Near end connector strains, and c) Midspan concrete strains.

Table 4.1 Material properties used in both the experimental and the numerical tests.

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Compressive strength (Mpa)</td>
<td>33</td>
</tr>
<tr>
<td>Steel beam*</td>
<td>YIELD strength (Mpa)</td>
<td>345</td>
</tr>
<tr>
<td>Studs*</td>
<td></td>
<td>355</td>
</tr>
<tr>
<td>Tendons*</td>
<td></td>
<td>1700</td>
</tr>
<tr>
<td>REINFORCEMENT bars*</td>
<td></td>
<td>300</td>
</tr>
</tbody>
</table>

*Young’s modulus: 200 Gpa, Poisson’s ratio: 0.3
4.5 Parametric Investigation

The experimentally verified numerical model was used to investigate the effect of various parameters on the monotonic behavior of the composite beams in the following sections. The parameters are variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the stud.

4.5.1 Effect of the degree of shear connection

The effect of variations in the degree of shear connection on the monotonic behavior of the strengthened composite beams was investigated. The degree of shear connection $\eta$ was taken as the ratio of the number of the studs used to the number of studs required to attain full shear connection. The behavior under four different $\eta$ values was compared, namely at 100% ($\eta=1$ ), 80% ($\eta=0.8$ ), 60% ($\eta=0.6$ ), and 40% ($\eta=0.4$ ). As can be seen in Fig. 4.6a, the stiffness and the ultimate load of the composite beams decrease as the degree of shear connection decreases. This could be due to the increase in stud connector strains as the degree of shear connection decreases, as shown in Fig. 4.6b. This increase in the stud strains was expected since the number of stud connectors that carry the shear forces decreases with $\eta$. The slippage at the interface between the concrete deck and the steel flange also increased with the degree of shear connection, as shown in Fig. 4.6c. It can be noted in Figures 4.6b and 4.6c that the effect of $\eta$ value on the stud strains and slippage
behavior increases as the value of η decreases. As the degree of shear connection was reduced from 100% to 80%, 60%, and 40%, the stud strains were increased by 20%, 46%,
and 94%, while the slippage values were increased by 23%, 48%, and 102%, respectively. Fig. 4.6d shows the increase in the tendon post-tensioning force as the beam is loaded. For all the shear connection degrees, the force in the tendon exhibited a slight increase up until the yielding load. After that, the tendon force increased significantly following the deflection response of the beams. Also, it can be seen in the figure that the overall increase in the tendon force, which is the difference between the PT force at the end of the test and the initial PT force, increases with the increase of shear connection degree. This increase in the tendon force could be attributed to the increased ductility associated with higher shear connection degrees.

### 4.5.2 Effect of layout and diameter of shear connectors

The effect of changing the arrangement of the stud connectors was also investigated in this study. The behavior of the composite beams was evaluated using two rows of studs (2R) and one row of studs (1R), as shown in Fig. 4.7, with full shear connection (η=1). Besides, four different stud diameters were used to evaluate the effect of changing the stud diameter, d, under each of the two arrangements evaluated, as shown in Table 4.2. The investigated diameters range from 12.7 mm to 25.4 mm. The total number of studs needed to attain full shear connection varied according to the diameter of the stud. Also, the spacing, S, between the studs is determined based on the arrangement and number of studs. For beams with two rows of studs, the spacings between the studs were twice the stud spacings for beams that have only one row of studs to maintain the same degree of shear connection. For each sample, the spacings were kept the same along the whole length of the beam. The load-deflection and load-slippage curves were compared for each of the different arrangement
types and diameters, as shown in Fig. 4.8. The load-deflection curves for 2R and 1R beams using various diameters are shown in Fig. 4.8a. As can be observed from the figure, for both 2R and 1R beams, the flexural capacity increases as the diameter of the stud decreases. Also, for each of the stud diameters, the 1R beams exhibited more flexural capacity than the 2R beams. It is worth noting that the spacings of the studs for 2R beams were double that for 1R beams to maintain the same shear connection level. Also, the difference between the ultimate loads between 1R and 2R beams using the same stud diameter increase as the diameter decreases. Fig. 4.8b shows the slippage values for the two arrangement types with different diameters. At all load levels, interface slippage increased significantly as the stud diameter increased. On the other hand, the effect of the arrangement type (2R vs. 1R) on slippage was insignificant. Fig. 4.8c compares the tendon force increase for the tested beams. For both 1R and 2R beams, beams with stud diameters of 12.7 and 15.875 mm experienced a 45% overall increase in the tendon force, on average. On the other hand, beams with stud diameters of 22.22 and 25.4 mm experienced an average increase in the PT force of 20%.

### 4.5.3 Effect of initial post-tensioning force

To investigate the effect of the post-tensioning (PT) force on the monotonic behavior of the strengthened beams, three initial PT force values were applied to the ends of the steel tendons. The PT forces were selected to be 55, 85, and 120 kN, which represent 30%, 45%, and 65% of the tendon’s ultimate strength, respectively. These values are within AASHTO’s maximum applied tendon force limit, which is 70% of the tendon’s strength. The responses of these strengthened beams were compared to a non-strengthened beam.
The deflection and the strains in the bottom flange of the steel beam at midspan as the monotonic load increases are shown in Fig. 4.9. Fig. 4.9a shows an increase in the ultimate load as the initial post-tensioning force increases. The maximum load for the beams with PTs of 55, 85, and 120 kN were 13%, 17%, and 23% higher than the ultimate load of the non-strengthened beam. Also, the increase in the ultimate capacity was 4%.

**Fig. 4.6** Effect of degree of shear connection; a) Load-deflection, b) Connector strain, c) Slippage, d) Tendon force.
Fig. 4.7 Top view of the steel beams showing the distribution of the studs.

Fig. 4.8 Effect of stud connector arrangement (1R vs. 2R) and diameter d (in mm) on a) Deflection, b) Slippage and c) Tendon force
when the PT force increased from 55 to 85 kN (55% increase in PT force) and was 5% when the PT force increased from 85 to 120 kN (41% increase in PT force). The deflection of the strengthened beam at failure was increased by 60% compared to that of the non-strengthened beam. This indicates a considerable improvement in the beam ductility due to post-tensioning. The increase in the ultimate capacities of strengthened samples is thought to be due to the reduced strains in the bottom flanges of the steel beams of strengthened samples at all load levels as the post-tensioning force increases. Fig. 4.9b shows the deflection-steel strain curves at the midspan of the beams. The strains in the steel beams at midspan for all the samples increase linearly until the nonlinear behavior started due to the development of the concrete section softening and the plastic hardening of the steel section. When the induced deflection level caused tension yielding to start and spread on the bottom side of the steel section, the steel strain started to plateau as shown in Fig. 4.9b. As the induced deflections in the samples exceeded the level at which the nonlinear behavior starts, the size of the damaged zone increases and localizes at the midspan, shifting the neutral axis of the composite section toward the neutral axis of the steel section. This caused a reduction in the rate of increase of tensile strains even after the plastic deformation as can be seen in Fig. 4.9c.

The effect of the post-tensioning force on tensile strains is observed when comparing the different PT forces used in Fig. 4.9c. At any given load level, the higher the PT force the lower the tensile strains. Also, compared to the non-strengthened beam, the midspan tensile strains at failure were reduced by 4%, 7%, and 15% for the beams that are strengthened with 55, 85, and 120 kN PT forces, respectively. The increase in the PT force during
loading for the three strengthened beams is shown in Fig. 4.9d. The PT force increased from 55, 85, and 120 kN to 90, 123, and 160 kN; respectively. These values represent a 64%, 45%, and 33% increase for the beams with initial PT of 55 kN, 85 kN, and 120 kN, respectively. This means that as the initial PT force increases, the overall increase in the tendon force decreases. This PT force response could be the reason why the effect of initial PT force on the load-deflection behavior was insignificant. That is, even after increasing the initial PT force from 55 kN to 120 kN (118% increase), the ultimate load and deflection were only slightly improved by less than 9%.

### 4.5.4 Effect of the steel beam depth.

The effect of the overall depth of the steel beam, D, on the behavior of the strengthened composite girders was investigated. Also, since keeping the distance e, which is the distance from the bottom face of the tension flange to the centroid of the post-tensioning tendons, the same while increasing the depth of the steel beams will result in increasing the eccentricity of the post-tensioning tendons, E (see Fig. 4.10), the effect of tendon eccentricity is also investigated. Steel beam sections with three different total depths were selected from the AISC’s Steel Construction Manual (“Steel Construction Manual” 2017); namely W10x30, W12x30, and W14x30. The sections were selected such that they have similar weights and areas. Five beams were tested for this investigation, which were labeled according to the depth of beam D and the eccentricity of the tendon E as shown in Table 4.3.

Figures 4.11(a), 4.11(b), and 4.11(c) show the behavior of the tested beams in terms of the load-deflection, steel strain-deflection, and load-steel strain results, respectively. The effect
of the steel beam depth is investigated by comparing samples D10E0, D12E0, and W14E0 since they have the same tenon eccentricity of 275 mm. As can be seen in Fig. 4.11(a), the load capacity of these beams increases as the steel beam depth increases. Compared to sample D10E0, the ultimate capacities of samples D12E0 and D14E0 were increased by 21% and 33%, respectively. The combined effect of steel depth and tendon eccentricity is investigated by comparing samples D10E0, D12E1, and W14E2 since their steel beam depths were varied with tendon eccentricities of 275 mm, 320 mm, and 360 mm, respectively. Compared to sample D10E0, the ultimate capacities of samples D12E1 and W14E2 were increased by 28% and 42%, respectively. Also, for samples with the same steel depth, the ultimate capacity increased by 6% when the tendon eccentricity was increased by 17% (sample D12E0 compared to sample D12E1).
Fig. 4.9 Effect of initial post-tensioning force; a) Load-deflection, b) Deflection-steel strain, c) Load-steel strain, and d) Load-tendon force.

Table 4.2 Number of studs and spacings for different arrangement types and diameters.

<table>
<thead>
<tr>
<th>Diameter mm (in.)</th>
<th>Number of studs</th>
<th>Spacing, S (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2R**</td>
</tr>
<tr>
<td>12.7 (½)</td>
<td>116</td>
<td>77</td>
</tr>
<tr>
<td>15.875 (5/8)</td>
<td>74</td>
<td>122</td>
</tr>
<tr>
<td>22.22 (7/8)</td>
<td>38</td>
<td>232</td>
</tr>
<tr>
<td>25.4 (1)</td>
<td>30</td>
<td>314</td>
</tr>
</tbody>
</table>

*One row of studs; **Two rows of studs

Fig. 4.10 Schematics showing the varying dimensions.
For samples D14E0 and W14E2, the eccentricity was increased by 31%, which resulted in an increase in the ultimate capacity by 7%. The same trend was exhibited in the samples in terms of the steel strain, as shown in Fig. 4.11(b). The higher the steel depth, the lower the midspan tensile strains. Also, samples with higher tendon eccentricity values performed better than those with lower values at the same steel depth. The general behavior of all samples in terms of midspan tensile strains, shown in Fig. 4.11(c), is similar to that of Fig. 4.9(c) where it linearly increases until the localization of the damaged zone occurs, then the rate of increase of tensile strains decreases significantly due to the shifting of the neutral axis as discussed previously. Fig. 4.11(d) is a comparison of the tendon force under the monotonic loading. As can be seen in the figure, the overall increase was similar for the beams that have the steel beam depth as the only varying factor (D10E0, D12E0, and D14E0). However, when the increase of the steel beam depth is accompanied by an increase in the tendon eccentricity, the overall increase in the PT force becomes significant, reaching to 47% increase.

4.5.5 Effect of the compressive strength of concrete.

Three samples with concrete compressive strengths ($f_c$) of 30, 40, and 50 MPa were tested to investigate the effect of the compressive strength of concrete on the flexural behavior of the strengthened composite girders. The load-deflection and the slippage behavior of these girders were compared and are presented in Fig. 4.12. The load-deflection response of the samples was the same until a load level of 350 kN. After that, an enhancement in the ultimate load and ultimate deflection was exhibited with the increase of the compressive strength, as shown in Fig. 4.12(a). Each increase of 11 MPa in the compressive strength
increased the ultimate load by an average of 6% and the ultimate deflection by an average of 12%. Fig. 4.12(b) shows the slippage behavior of the samples as the load increases. The overall slippage was reduced by an average of 11% as the compressive strength of concrete increased by 10 MPa. The increase of the concrete compressive strength had almost no effect on the PT force in the tendon as all beams experienced a similar overall increase as shown in Fig. 4.12(c).

4.5.6 Effect of degradation of stud connectors

The shear capacity of studs can be affected by corrosion, fatigue, weld defects, or unexpected overloading. These damages cause degradation in the stud shear capacity. Therefore, an investigation on the effect of stud shear capacity on the behavior of the strengthened composite girders was conducted. The design capacity of the shear connector, $P_u$, in the load-slip relationship presented in this study in equation (2) was used as a parameter to investigate the effect of the shear capacity of studs on the overall performance of the strengthened composite beams. Three values of $P_u$ were considered, namely 90 kN, 45 kN, and 23 kN. The load-deflection results are presented in Fig. 4.13(a). A 12% reduction in the ultimate load was noticed when the stud shear capacity was reduced from 90 to 45 kN (50% capacity reduction). The composite beam ultimate load was reduced by 24% when the stud shear capacity was reduced by 75% (from 90 to 23 kN). The slippage behavior is shown in Fig. 4.13(b). The overall slippage increased by 80% and 95% when the stud capacity was reduced by 50% and 75%, respectively.
Fig. 4.11 Effect of steel beam size; a) Load-deflection, b) Steel strain-deflection c) Load-steel strain, and d) Load-tendon force.
Fig. 4.12 Effect of concrete compressive strength; a) Deflection, b) Slippage, and c) Tendon force

Fig. 4.13c shows the increase in the PT force for the three beams. The initial PT forces were increased by 45%, 35%, and 18% for the beams with $P_u$ of 90 kN, 45 kN, and 23 kN, respectively. This indicates that the response of the tendon force is significantly affected
by the degradation of shear capacity. Also, as can be seen in the figure, as the $P_u$ value decreases, the reduction in the PT force increases. This

**Table 4.3** Test matrix for investigating the effect of the steel beam depth and tendon eccentricity.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description of the sample</th>
<th>Section</th>
<th>$E$ (mm)</th>
<th>$e$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D10E0</td>
<td>Depth of the steel beam, D, is 10” &amp; $E=275$ mm</td>
<td>W10×30</td>
<td>275</td>
<td>32</td>
</tr>
<tr>
<td>D12E1</td>
<td>D is 12” &amp; $E=320$ mm</td>
<td>W12×30</td>
<td>320</td>
<td>32</td>
</tr>
<tr>
<td>D14E2</td>
<td>D is 14” &amp; $E=360$ mm</td>
<td>W14×30</td>
<td>360</td>
<td>32</td>
</tr>
<tr>
<td>D12E0</td>
<td>D is 12” &amp; $E=275$ mm</td>
<td>W12×30</td>
<td>275</td>
<td>78</td>
</tr>
<tr>
<td>D14E0</td>
<td>D is 14” &amp; $E=275$ mm</td>
<td>W14×30</td>
<td>275</td>
<td>117</td>
</tr>
</tbody>
</table>

PT force response is consistent with load-deflection and slippage responses. Thus, it could be concluded from these results that as the shear capacity of the studs decreases, its effect on the monotonic beam behavior also decreases.

### 4.6 Summary and Conclusions

In this study, a three-dimensional finite element model was developed to simulate the monotonic behavior of externally strengthened steel-concrete composite girders. The results obtained from the model were validated using the results of experimental tests that were previously conducted by the authors. After that, a parametric study was performed to investigate the effect of several parameters on the flexural performance of the strengthened composite girders. The parameters investigated in this study are variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of
concrete, and the shear capacity of the stud. Several observations from the results of this study are listed below.

**Fig. 4.13** Effect of degradation of stud connector; a) Deflection, b) Steel strain, c) Tendon force.
• Variations in shear connection degree greatly affect the behavior of the strengthened composite beams. The stiffness and the ultimate load capacity of the beams decreased as the shear connection degree decreased. Also, stud strains and the interface slippage increased as the shear connection degree decreased.

• As the diameter of the studs increased, the flexural capacity decreased, and the interface slippage increased. Also, for the same shear connection degree, the distribution of the studs in one row rather than two rows improved the flexural capacity while having no significant impact on the slippage behavior.

• The deflection of the strengthened beam at failure was increased by up to 60% compared to that of the non-strengthened beam. Strengthening the samples resulted in a significant increase in the ultimate load with a reduction in the tensile strains at midspan.

• The load capacity increases as the steel beam depth increases regardless of the tendon eccentricity. Also, for the same steel depth, the ultimate capacity slightly increases as the tendon eccentricity increases. The best performances in terms of the ultimate capacity and tensile strains were obtained when the increase in the steel depth was accompanied by an increase in the tendon eccentricity.

• Increasing the concrete compressive strength resulted in increased ultimate load, increased ultimate deflection, and reduced overall slippage.

• The shear capacity of studs affected the overall behavior of the composite beams. The ultimate load decreases, and the slippage increases as the shear capacity decreases.
4.7 References


Uy, Brian, and Stewart Craine. 2004. “Static Flexural Behaviour of Externally Post-


Chapter 5: Progressive Fracture of Shear Connectors on the Residual Response of Steel-Concrete Composite Girders

5.1 Abstract

Prestressed steel-concrete composite beams are widely used in bridges around the world. Loads throughout the service life of bridges may cause failure in the form of fracture in the studs near the ends of the bridge girders. The effect of the stud failure on the residual static capacity and the residual fatigue life of the composite girders is not well investigated. Therefore, this study presents a numerical investigation on the residual behavior of prestressed composite beams with fractured studs at the end regions of the beams. The finite element model was validated by using existing experimental work. The objective of this study is to investigate the effects of the progressive failure of stud shear connectors on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. Also, the effect of stud fracture on the steel-concrete interface slippage, shear stress range and compressive and tensile strains were studied. The behavior of the composite girders in terms of the estimated fatigue life and residual capacity was affected by the number of removed studs. Until 15% of the rows were removed, the strengthened sample had a better response in terms of the tensile and compressive strains and residual ultimate load. The effect of stud fracture on the shear stress ranges experienced by the shear connectors mainly manifested in the beam ends where stud fracture occurred. Also, neglecting the steel-concrete interface slippage in the theoretical calculations of stud shear
stress ranges resulted in a significant underestimation of the shear stress ranges experienced by the shear connectors.

5.2 Introduction

Steel-reinforced concrete composite girders are the main structural elements for many bridges around the world. The advantages of the rapid construction of steel girders and the cost-effectiveness of the reinforced concrete slab made it an attractive option for bridge constructions. External prestressing has been used in steel-reinforced concrete composite girders for decades owing to its ease of application. Using this prestressing method has been shown to be effective to upgrade the flexural capacity of composite beams (Reagan, R.S. and Krahl 1967). The behavior of prestressed steel-reinforced concrete girders is greatly affected by the existence of shear connectors, a key component that plays a vital role in preventing slippage and separation between the concrete slab and the steel girder. The degree of shear connection provided by the shear connectors greatly affects the structural performance and failure mode of composite girders (Sanghyo, Chiyoung, and Jinhee 2011). Headed studs are the most used type of shear connectors in steel-concrete composite girders.

In addition to the static load, composite bridge members are prone to damage from traffic-induced cyclic loading, which is very detrimental to the integrity of the shear connectors, and hence, the structural integrity of the overall composite girder. Repeated loads can lead to the initiation of microcracks in the shear connectors, which propagate as the cyclic loading continues until the failure of the shear connector. This failure process, known as fatigue failure, happens suddenly with no significant deformations preceding it (Mia and
One of the most common fatigue failure modes of stud shear connectors is the fracture at the base of the stud. This failure mode usually starts in composite girders in the studs located in the beam end regions (Wang et al. 2018). The fracture of studs in only one end of the beam was reported by Wang et al. (2021) as the failure mode of a fatigued steel-concrete composite beam. Other common failure modes include cracking of the steel beam around the studs and cracking in the welding zone of the studs, both of which expanding until the complete fracture of studs, as reported by Yu-Hang et al. (2014). All these failure modes are the consequences of cracks that initiate due to stress concentration and imperfections in the materials used, which propagate during loading until failure of shear connectors and affect the static capacity and life span of the global composite beams.

Limited research exists on the effects of stud failure on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. The impact of crack initiation and propagation in studs on the static and fatigue behavior of the steel-concrete composite beam was studied experimentally and numerically by Hanswille et al. (2009). They concluded that the residual load-deflection behavior was significantly affected by the cracking of the studs and that the reduction of the composite action due to stud cracking caused an 8% reduction in the maximum load capacity of the global steel-concrete beam. Yu-Hang et al. (2014) conducted experimental tests where fatigue loading was applied until failure in the form of fractured studs at the end regions of the tested specimens occurred, then the specimens were loaded under static loading to evaluate their residual static performance. The failure modes of all the tested beams were reported to be shear fatigue failure of the studs. In a recent experimental study, conducted by Wang et al.
(2021), the residual static capacity of composite beams were evaluated after being subjected to various number of fatigue load cycles. They observed that fracture of the studs at the end regions of the beams increases with the increase of the number of fatigue cycles experienced before the static test. This caused the residual static capacity to be reduced to be only 70% of the capacity of a reference sample with intact studs. Also, the ultimate deflection of the composite beam with fractured studs was reduced to only 40% of its reference value.

The current study is an effort to contribute to the understanding of the effect of stud fracture on the behavior of steel-reinforced concrete composite girders. Therefore, the objective of this study is to investigate the effects of the failure of stud shear connectors on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. Also, the effect of stud fracture on the steel-concrete interface slippage, shear stress range and compressive and tensile strains were studied. Due to the cost and time-consuming nature of the experimental investigation of steel-concrete composite girders, a finite element model was built to simulate this failure scenario. This study presents a numerical investigation of the residual behavior of prestressed composite beams with fractured studs at the end regions of the beams. The study was done using a finite element model that was validated using three experimental works available in the literature. The development of the finite element model is discussed next.

### 5.3 Finite Element Modeling

The finite element (FE) software Ansys (v. 18.1) was used to develop the numerical models of steel-reinforced concrete girders investigated in this paper. The developed numerical
models adopted both geometric and material nonlinearities. Moreover, the developed FE models simulated each component of the strengthened composite beams. These components included web and flanges of the steel beam, concrete deck, headed stud shear connectors, steel reinforcement, and externally post-tensioned tendons. Models of three experimentally tested composite samples were built. The description of these samples and the experimental details are presented in the model validation section.

### 5.3.1 Element selection

8-nodes solid element, SOLID185, was used to model the steel webs and flanges of the steel beams. This element has three degrees of freedom at each node: translations in the x, y, and z directions. Moreover, its capabilities include plasticity, stress stiffening, large deflection, and large strain. To consider the capability of crushing in compression and cracking in tension, the concrete decks were modeled using the SOLID65 element. This element has three degrees of freedom at each node: translations in the x, y, and z directions. The shank of the headed stud shear connectors was modeled using beam element BEAM188, which has six degrees of freedom at each node. These include translations in the x, y, and z directions and rotations about the x, y, and z directions. The longitudinal and transverse deck reinforcements as well as the post-tensioned tendons were modeled using a 2-node link element, LINK180. The composite interaction between the steel beams and the concrete decks was simulated using two different elements. The non-linear spring element, COMBIN39, simulated the relative movement between the two surfaces in contact. To prevent any physical penetration between the two surfaces, 8-node surface-to-
surface contact elements, CONTAC174, were used. An example of the loading setup and details of the different components of one of the composite beams are presented in Fig. 5.1.

5.3.2 Real constants

To define the initial post-tensioning force \( F \) of strengthened samples, an equivalent drop in temperature based on equation (1) was defined.

\[
F = EA \alpha \Delta T
\]

(1)

where \( E \) is the modulus of elasticity, \( A \) is the cross-section area of the tendon, \( \alpha \) is the coefficient of thermal expansion (10e\(^{-5}\) for steel), and \( \Delta T \) is the temperature drop.

Convergence studies were performed to choose the FE mesh that provides accurate results with minimum computational time. The boundary conditions were modeled as simply supported beams, which were the same as those of the experimental test setups.

The exponential equation developed by Ollgaard et al. (1971) was used to represent the relative movement between the concrete decks and the steel beams (Equation 2)

\[
P = P_u (1 - e^{-\beta \delta})^\gamma
\]

(2)

where \( P \) is the horizontal shear force in the headed stud shear connector, \( P_u \) is the shear capacity of the headed stud shear connector, \( \delta \) is the interface slippage between the steel beam and concrete deck, and \( \beta \) and \( \gamma \) are coefficients related to the stiffness of the studs.
5.3.3 Constitutive models of materials

The tensile behavior of concrete was assumed to be linear up to failure and then progressively was reduced to zero. On the other hand, the compressive behavior of concrete was simulated by a uniaxial stress-strain relationship. The bilinear model, which is adopted in ANSYS, was used to simulate the behavior of steel reinforcement. For the linear isotropic part, it is defined by the modulus of elasticity of the reinforcement and the Poisson’s ratio, and away from this point, strain hardening was defined beyond the yield point. The material properties used in the FE model for each beam were identical to those used in the corresponding experimental tests, which will be described in the model validation section.

5.4 Validation of the 3D FE Model

The proposed finite element model was validated against the results of three existing experimental tests on simply supported steel-concrete composite beams. Details of the dimensions of the cross-sections of the three beams are shown in Fig. 5.2, and their values are provided for each beam in
Table 5.1. The first beam (B1) was a non-strengthened 4-meter-long steel-reinforced concrete composite girder, which was experimentally tested by Prakash et al. (2012). The static loading was applied at two points that are 600 mm apart with each point being 300 mm from midspan. The second beam (B2) was a strengthened beam using post-tensioning tendons with a support span of 5.524 meters tested by Lorenc and Kubica (2006). A cover plate that is 4.5 meters long is welded to the bottom flange of the steel beam starting from

![Fig. 5.1 The loading setup and details of the different components of the composite beam a) longitudinal direction b) transverse direction showing the deck reinforcement.](image-url)

104
0.512 meters from the supports. The static test was set as a four-point bending test with a moment span of 1 meter. The third beam (B3) was a strengthened beam using post-tensioning tendons with a support span of 4.42 meters, tested by Alsharari et al. (Alsharari et al. 2021). This beam was loaded at a single point at midspan. Additional details of the different components of the composite girders are provided in Table 5.2.

Fig. 5.3 shows the load-deflection curves of the composite girders tested experimentally and compares them to those obtained using the developed FE models. The load-deflection curves for B1 are shown in Fig. 5.3a. The FE model almost perfectly matched the FE model up until the maximum load of the experimental curve. After that, the FE model curve was 18% higher, on average, than the experimental curve. However, the difference between the ultimate loads of both curves was within 4% and the difference between the ultimate deflections was within 10%. Fig 5.3b shows the load-deflection curves for B2. The FE model results were representative of the experimental results except that the FE model results were slightly stiffer and predicted a 7% higher ultimate load. Fig. 5.3c shows the load-deflection curves for beam B3. The FE model results were consistent with the experimental load-deflection curve except that, in the linear part of the curve, the FE model predicted slightly higher stiffness than the experimental test. Also, the maximum load of the FE model and experimental test were consistent.

5.5 Simulating the Progressive Failure of Studs

The experimentally validated FE model developed in this paper was used to investigate the progressive failure of stud shear connectors. Beam B3 was selected for the investigation of the
Table 5.1 Dimensions of the cross-sections of the three beams

<table>
<thead>
<tr>
<th>Sample</th>
<th>Wd</th>
<th>hd</th>
<th>hf</th>
<th>bf</th>
<th>bw</th>
<th>hs</th>
<th>hc</th>
<th>bc</th>
<th>e</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>665</td>
<td>150</td>
<td>11.8</td>
<td>165.7</td>
<td>6.7</td>
<td>306.6</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>B2</td>
<td>800</td>
<td>100</td>
<td>13</td>
<td>170</td>
<td>8</td>
<td>360</td>
<td>10</td>
<td>150</td>
<td>46</td>
</tr>
<tr>
<td>B3</td>
<td>1145</td>
<td>101</td>
<td>13</td>
<td>148</td>
<td>7.5</td>
<td>266</td>
<td>NA</td>
<td>NA</td>
<td>19</td>
</tr>
</tbody>
</table>

Table 5.2 Details of the different components of the composite girders

<table>
<thead>
<tr>
<th>geometry</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of studs, mm</td>
<td>100</td>
<td>75</td>
<td>80</td>
</tr>
<tr>
<td>Longitudinal spacing of studs, mm</td>
<td>283</td>
<td>100</td>
<td>122</td>
</tr>
<tr>
<td>Support span of beam, mm</td>
<td>3800</td>
<td>5524</td>
<td>4420</td>
</tr>
<tr>
<td>Stud’s diameter, mm</td>
<td>20</td>
<td>13</td>
<td>15.875</td>
</tr>
<tr>
<td>Diameter of reinforcing bars, mm</td>
<td>12</td>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>Tendon’s cross-section area, mm²</td>
<td>NA</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td>Initial prestressing force per tendon, kN</td>
<td>NA</td>
<td>145</td>
<td>85</td>
</tr>
<tr>
<td>Compressive strength of concrete, MPa</td>
<td>36</td>
<td>32</td>
<td>33</td>
</tr>
<tr>
<td>Yield strength Fy &amp; ultimate strength Fu, MPa</td>
<td>Fy</td>
<td>Fu</td>
<td>Fy</td>
</tr>
<tr>
<td>Bars</td>
<td>415</td>
<td>550</td>
<td>428</td>
</tr>
<tr>
<td>Tendons</td>
<td>NA</td>
<td>NA</td>
<td>1600</td>
</tr>
<tr>
<td>Steel beam section</td>
<td>300</td>
<td>420</td>
<td>300</td>
</tr>
<tr>
<td>Headed studs</td>
<td>680</td>
<td>900</td>
<td>515</td>
</tr>
</tbody>
</table>
effect of fracture of shear studs on the residual capacity and the remaining fatigue life of steel-reinforced concrete composite girders. The progressive fracture of the studs was simulated by successive removal of rows of shear studs from one end of the beam. To facilitate evaluating specific shear stud rows, the rows were numbered starting from the end from which the studs were removed as shown in Fig. 5.4. A preliminary study was

Fig. 5.3 The FE and Experimental results for the load-deflection curves for a) sample B1 b) sample B2 and c) sample B3
conducted, and it was shown that after 10 rows were removed, the applied shear force of the end studs exceeded their ultimate capacity. Therefore, the stud removal study was limited to the first 10 rows. The removal of the shear studs was done in 10 successive steps by removing one row (two studs) per step (see Fig. 5.5) until a total of 10 rows were removed.

After each row was removed, the beam was subjected to two loading types. The first type is loading the beam with a load amplitude of 140 kN to simulate a fatigue load cycle, and the second type is loading the beam until failure to evaluate its residual capacity. Since damage of shear studs in composite bridge members is more likely to be caused by fatigue cycles in load levels below the yield strength of the beams. The samples were loaded with a minimum and a maximum load of 10 kN and 150 kN, respectively, which represent a fatigue cycle whose maximum load is approximately 40% of the load-carrying capacity of a reference sample. These loading values were chosen such that the load amplitude used in this study is similar to fatigue load amplitudes experimentally applied to similar beams that were tested for a different study by Alsharari et al. (2021). The sample was loaded at a single loading point at midspan on a 200mm×200mm×35mm steel plate. It is worth noting that the fracture of stud shear connectors, in practice, is more likely to be accompanied by degradation in the strengths of the different components of the composite beams. However, this study focused on the effect of the fracture of studs alone on the behavior of composite beams.
5.6 Results

The effects of the progressive failure of stud shear connectors on the steel-concrete interface slippage, shear force, shear stress range, compressive and tensile strains at midspan, and the remaining fatigue life of studs were evaluated under a load amplitude of 140 kN for beam B3. In addition, the residual capacity of the composite beams was evaluated by loading until failure. A non-strengthened version of the beam B3 was also modeled, by removing the post-tensioning tendons, to evaluate the effect of strengthening on the responses of the beams. For ease of identification, the strengthened sample is labeled as S while the non-strengthened sample is labeled as NS.
5.6.1 Effect on the slippage

The slippage at the steel-concrete interface at the beam ends (see Fig. 5.6) was monitored after the removal of each row. The slippage values corresponding to the maximum load were taken after each step from each beam end, and the results are presented in Fig. 5.7 for both the strengthened and non-strengthened samples. The slippage values at the side from which the studs were removed are shown in Fig. 5.7a and the slippage values at the side where no studs were removed are shown in Fig. 5.7b. The slippage values of the removal side increased significantly as the number of the removed studs increased while almost no change occurred in the slippage values on the non-removal side. The lack of studs at the beam ends to resist the horizontal movement between the concrete deck and the steel beam caused the higher slippage values and incremental slippage increase at the removal side. The response of the strengthened and non-strengthened samples was similar in both beam ends, with the slippage values for the strengthened sample being slightly lower than the non-strengthened sample.

![Fig. 5.6 Slippage example in the steel-concrete interface region at the beam end.](image)
5.6.2 Effect on the stud shear forces.

To evaluate the effect of the progressive failure of studs at one end of the beam on the shear force experienced by the shear studs, the results of the maximum and minimum shear forces in the most critical region of the composite beam were monitored. Since this study involves removing the first ten rows, the eleventh row (#11 in Fig. 5.4) is the closest non-removed row to the beam ends, which experiences the highest slippage values. Thus, the results of the eleventh row were used for this study. The maximum and minimum stud shear force values were taken after each row was removed for both the S and NS samples. Fig. 5.8a shows the maximum shear force results as the number of removed rows increased. The increase in the maximum shear forces for both S and NS samples was slow until the removal of the fifth row, after which the rate of change significantly increased causing the maximum shear force to reach 63 kN after 10 rows were removed.

![Slippage Graph](image)

**Fig. 5.7** Effect of progressive failure on the interface slippage; (a) removal side, (b) non-removal side.
removed, which represents approximately 70% of the stud shear resistance. The reduction in the maximum shear force due to post-tensioning, which is the difference in the maximum shear force between the strengthened and non-strengthened beam, increased as the number of removed rows increased. However, the difference was within 1 kN, as can be seen in Fig. 5.8b. Fig. 5.9 shows the minimum shear force results for the S and NS samples. Like the maximum shear force behavior, the minimum shear force gradually and slightly increased until 5 rows were removed. After that, the rate of change in the minimum shear force sharply increased with the strengthened sample having 1 kN less minimum shear force compared to the non-strengthened sample.

![Graphs showing Stud fracture effect on the maximum shear force](image)

**Fig. 5.8** Stud fracture effect on the maximum shear force
Effect on the shear stress range across the beam

As shown in the previous section, the maximum and minimum shear force increased at a similar rate for samples S and NS, hence the shear stress range for both the strengthened and non-strengthened samples was the same and was not affected by the post-tensioning. Therefore, the shear stress range results across the composite beam after each step are presented here for the strengthened sample only. The shear stress range applied to the studs across the strengthened composite beam was taken after each step to evaluate the effect of the stud removal on stress ranges of both sides of the beams. Fig. 5.10 shows a comparison of the shear stress range results when no rows were removed (step 0) and the shear stress range results after steps 2, 5, 8, and 10. Although theoretically the shear stress is expected to be constant between the loading point and the beam end, the results obtained from the model show that the end regions of the beam have higher stress ranges than the midspan region even before any studs were removed. After each removal, the non-removed row

![Graph showing the effect of stud removal on minimum shear force](image)

Fig. 5.9 Stud fractur effect on the minimum shear force

5.6.3 Effect on the shear stress range across the beam

As shown in the previous section, the maximum and minimum shear force increased at a similar rate for samples S and NS, hence the shear stress range for both the strengthened and non-strengthened samples was the same and was not affected by the post-tensioning. Therefore, the shear stress range results across the composite beam after each step are presented here for the strengthened sample only. The shear stress range applied to the studs across the strengthened composite beam was taken after each step to evaluate the effect of the stud removal on stress ranges of both sides of the beams. Fig. 5.10 shows a comparison of the shear stress range results when no rows were removed (step 0) and the shear stress range results after steps 2, 5, 8, and 10. Although theoretically the shear stress is expected to be constant between the loading point and the beam end, the results obtained from the model show that the end regions of the beam have higher stress ranges than the midspan region even before any studs were removed. After each removal, the non-removed row
closest to the removed studs experienced the highest stress ranges with the stress range gradually decreasing toward the midspan, after which the effect of stud removal is negligible. As can be seen in Fig. 5.10, the shear stress range at any given location increased as the number of removed rows increased. This is attributed to the high steel-concrete interface slippage experienced by the beams at the removal side. This slippage, and consequently the shear stress range, increased as the number of removed studs increased as discussed previously (see Fig. 5.7a). The stress ranges at the non-removal side were almost the same after each step, which is also consistent with the constant slippage values shown in Fig. 5.7b.

![Shear stress range across the composite beam after removal of studs.](image)

**Fig. 5.10** Shear stress range across the composite beam after removal of studs.
5.6.4 Effect on the remaining fatigue life

The number of fatigue cycles until failure of the shear stud under the applied shear stress range can be calculated using AASHTO’s S-N curve equation (AASHTO 2012), which relates the number of remaining fatigue cycles, \(N\), until failure of the stud to the shear stress range, \(\Delta\tau\), in the studs as follows:

\[
\log N = 8.061 - 0.0266 \times \Delta\tau
\]  

(3)

In this section, the theoretically calculated shear stress ranges of studs in the eleventh row are compared to those obtained from the FE model. Then, using Equation 3, the corresponding remaining fatigue life is evaluated for the theoretical and FE results. The theoretical calculation of shear stress ranges involved using the shear flow equation and properties and dimensions of the sample as in Equation 4:

\[
\Delta\tau = \frac{q_s s}{A_s}
\]

(4)

Where \(q\) is the shear flow (unit force per unit length) calculated using Equation 5, \(s\) is the spacing between studs, and \(A_s\) is the area of shear connectors.

\[
q = \frac{V Q}{I}
\]

(5)

where \(V\) is the shear force, \(Q\) is the first moment of the area for the concrete deck about the neutral axis of the cross-section, and \(I\) is the second moment of inertia. Fig. 5.11a shows the comparison between the theoretical shear stress range after each step and the FE results. As shown in the figure, the theoretical shear stress range increased linearly after each row.
removal with a constant and relatively low rate of change. The shear stress range was 75 MPa when no studs were removed, and it was doubled after the removal of 10 rows. The FE shear stress results, on the other hand, show that the shear stress range was slightly lower than the theoretical values until 5 rows are removed. After that, the shear stress range started to increase significantly reaching a maximum of 300 MPa after step 10. This could be because the theoretical calculation of the shear stress range does not consider the effect of the slippage and assumes a full connection between the steel and concrete. Therefore, until five rows of studs were removed, which represent 15% of the total number of rows, the effect of the slippage was not apparent, and the difference in stress ranges of the theoretical and FE results was insignificant. As the number of removed studs increases, which resulted in increased slippage, the FE results gave higher shear stress ranges than the theoretical values due to the inclusion of the slippage effect in the model.

The theoretical and FE shear stress values in Fig. 5.11a were substituted into Equation 3 to calculate the estimated remaining fatigue life of the stud. Fig. 5.11b shows the results of the remaining fatigue cycles after each removal step. As can be seen in the figure, when no studs were removed, the number of remaining fatigue cycles was theoretically expected to be 1.135 million cycles while the FE model is suggesting a fatigue life of 1.62 million cycles, which is 42% more than the theoretical result. Before the number of removed rows reached 5, the FE results estimated higher remaining fatigue cycles than the theoretical results with a maximum difference of 75%. After that, the theoretical results were overestimating the fatigue strength of the studs, which is due to the effect of slippage being neglected in the theoretical calculations.
5.6.5 **Effect on midspan tensile and compressive strains**

The midspan tensile and compressive strains corresponding to the maximum load were taken after each stud removal step for both the strengthened and non-strengthened samples. Fig. 5.12a shows the tensile strains at the bottom flange of the steel beam at midspan as the number of removed stud rows increased. The effect of stud removal was more apparent in the non-strengthened beam where the rate of increase in strain is higher than that of the strengthened sample. The strengthened sample’s tensile strain was constant as the number of removed studs increased and only slightly increased after 8 rows were removed. The tensile strain for the non-strengthened sample increased by 2.5% from step 0 (when no studs were removed) to step 10 (when 10 rows were removed). The incremental increase
in the tensile strain of the non-strengthened sample was 175% higher than that of the strengthened sample.

Fig. 5.12b shows the compressive strains at the concrete deck at midspan as the removed studs increases. For the non-strengthened sample, the compressive strain increased linearly as the number of removed studs increased with a higher rate of change after 5 rows were removed. The compressive strains for the strengthened sample, on the other hand, were constant until 5 rows were removed, after which the strains started to increase at a rate comparable to the of the non-strengthened sample. The overall incremental increase in compressive strains was increased by 8% for the non-strengthened sample and 6% for the strengthened sample. The increase in the tensile and compressive strains is thought to be due to the loss of composite action between the concrete deck and the steel beam due to the separation of the two components at the removal side, causing each component to behave individually rather than compositely.

5.6.6 Effect on the flexural strength

To evaluate the reduction in the flexural capacity caused by the progressive removal of studs, the strengthened and non-strengthened samples were loaded until failure after each row was removed. Fig. 5.13 shows the maximum load-carrying capacity of each sample as the number of removed studs increases. As shown in the figure, the maximum load-carrying capacity for both S and NS beams decreased as the number of removed studs increased. The maximum load after 5 rows were removed was reduced by 8.5 kN for the non-strengthened sample and by 1.6 kN for the strengthened sample, which indicates a higher rate of reduction for the non-strengthened sample.
After that, however, the strengthened beam exhibited a higher rate of reduction. Between steps 5 and 10, the maximum load was reduced by 27 kN and 15 kN for S and NS samples, respectively. Fig. 5.14 shows the ratio of the maximum load after each row was removed.

Fig. 5.12 Effect of progressive failure on the midspan tensile strain (a) and midspan compressive strain (b).

Fig. 5.13 The maximum load-carrying capacity as the number of removed studs increases.

After that, however, the strengthened beam exhibited a higher rate of reduction. Between steps 5 and 10, the maximum load was reduced by 27 kN and 15 kN for S and NS samples, respectively. Fig. 5.14 shows the ratio of the maximum load after each row was removed.
to the maximum load of an intact beam (with no studs removed). Both S and NS samples experienced an overall 7% reduction in the maximum load after 10 rows were removed.

Fig. 5.15a shows the load-deflection curves for the strengthened and non-strengthened samples after steps 0, 5, and 10. The effect of stud removal on the maximum load capacity increased as the number of removed studs increased for both S and NS samples. The stiffness, yield strength, and load capacity of the composite beams decreased as the number of removed studs decreased. This could be attributed to the increase in shear force experienced by the remaining studs, as shown in Fig. 5.15b, that is caused by the loss of composite action.

5.7 Summary and Conclusions

This paper presents a numerical investigation on the progressive failure of stud shear connectors of prestressed steel-concrete composite beams with fractured studs at the end
regions of the beams. The study was done using a finite element model that was validated using existing experimental studies. The objective of this study was to investigate the effects of the progressive failure of stud shear connectors on the residual static performance and the remaining fatigue life of strengthened steel-concrete composite beams. Also, the effect of stud fracture on the steel-concrete slippage, shear stress range and compressive and tensile strains were studied. Several observations from this study are summarized below:

- The effect of stud fracture on the shear stress range of the shear connectors mainly manifested at the beam ends where stud fracture occurred.

- Neglecting the steel-concrete interface slippages in the theoretical calculation of stud shear stress ranges resulted in a significant underestimation of the shear stress ranges experienced by the shear connectors.

![Fig. 5.15 Load-deflection response (a) and increase in shear force-deflection response (b) for S and NS samples after steps 0, 5, and 10.](image)
• As studs fracture, the loss of composite action causes the maximum compressive and tensile strains at midspan to increase.

• The estimated remaining fatigue life of the studs was significantly affected by the number of removed studs. The AASHTO theoretical equation conservatively estimated the remaining fatigue life until 5 rows were removed, after that the FE model predicted lower remaining fatigue cycles than the theoretical ones.

• The residual flexural capacity for the non-strengthened sample decreased at a constant rate as studs were removed. However, the rate of reduction for the strengthened sample increased noticeably after 5 rows were removed.

• Until 15% of the rows were removed, the strengthened sample had a better response in terms of the stress range, tensile and compressive strains, and residual capacity. After that, both the strengthened and non-strengthened samples exhibited similar responses to the failure of studs.
5.8 References


Yu-Hang, Wang, Nie Jian-Guo, and Li Jian-Jun. 2014. “Study on Fatigue Property of Steel-
Chapter 6: Conclusions and Recommendations

6.1 Summary

Understanding the fatigue and static behavior of strengthened composite beams is vital since bridge structures are usually exposed to fatigue damage. Also, when strengthening an in-service bridge component, the effect of pre-fatigue conditions on the behavior of post-tensioned steel-concrete composite (SCC) beams should be considered. In this research, the fatigue and static behavior of steel-concrete composite girders externally strengthened with post-tensioning steel tendons were evaluated. Experimental and numerical tests were conducted to study the fatigue and static behavior under damaged and undamaged conditions. In addition, the progressive stud shear connector failure was evaluated numerically.

Five samples were experimentally tested under four-point fatigue loading. The effect of external post-tensioning (PT) as a retrofit on the cyclic crack patterns on the concrete flanges, cyclic incremental deformations, and strains were investigated with various pre-fatigue conditions. The pre-fatigue conditions included exposure to outdoor environmental changes for 365 days, plastic deformations, and fatigue damages. Static tests to failure were performed on the fatigued samples to evaluate their residual capacities. The tests were performed after the completion of the fatigue tests to compare their residual capacities, deformations, and strains. The failure modes of the steel-concrete composite beams were also evaluated.
The monotonic behavior of externally post-tensioned SCC girders was numerically studied in this research. A three-dimensional numerical model was developed and validated using the experimental test results presented in this research as well as experimental test results that are available in the literature. A parametric study using this validated numerical model was performed to investigate the effects of various parameters on the monotonic performance of composite girders strengthened with external post-tensioned tendons. The parameters investigated include variations in the degree of shear connection, layout and diameter of shear connectors, the initial post-tensioning force, the depth of the steel beam, the eccentricity of the tendons, the compressive strength of concrete, and the shear capacity of the studs.

The effect of progressive stud failure on the residual static capacity and the residual fatigue life of SCC girders was numerically investigated. Since fracture in the studs near the ends of bridge girders was reported as a common failure form in SCC girders, the objective of this study is to investigate the effect of the progressive failure of stud shear connectors on the residual static performance, remaining fatigue life, slippage, shear stress range, and compressive and tensile strains of strengthened steel-concrete composite beams.

6.2 Contributions

The main contributions of this research are:

- Experimentally investigated the effect of external PT on the performance of the different components of the steel-concrete composite section during fatigue.
• Expanded the state of knowledge by evaluating the effects of pre-existence of
damage on the performance of strengthened SCC beams.
• Evaluated the residual capacities of damaged and non-damaged fatigued steel-
concrete composite beams strengthened with externally post-tensioned steel
tendons.
• Developed a three-dimensional finite element (FE) model that can simulate
the behavior of composite steel-concrete girders strengthened with external
post-tensioned steel tendons.
• Performed parametric studies using the FE model to investigate the effects of
various important parameters on the flexural behavior of the strengthened
composite girders.
• Investigated the effect of progressive stud fracture on the behavior of
strengthened SCC girders.

6.3 Conclusions

• During experimental fatigue loading, the external PT caused reduction of the
cyclic strains in the concrete flanges, steel beams, and shear connectors.
However, longitudinal, and transverse cracks in the top surface of concrete were
initiated and propagated.
• Subjecting the samples to fatigue loading or environmental changes before
strengthening caused higher damages in concrete around the shear connectors
and led to more incremental deformations and strains relative to that when the sample was subjected to plastic deformation before strengthening.

- Subjecting the sample to plastic deformation before strengthening resulted in an improved fatigue performance, increased residual stiffness and ultimate load, and reduced ductility.

- The typical failure mode observed after the experimental residual static test for all samples was yielding in the steel beam followed by crushing in the concrete deck.

- Numerical investigation of the effect of the shear connection degree showed that the behavior of the strengthened composite beams was greatly affected by variations of the degree of the shear connection. The stiffness and the ultimate load capacity of the beams decreased as the shear connection degree decreased. Also, stud strains and the interface slippage increased as the shear connection degree decreased.

- The results of the numerical parametric study showed that as the diameter of the studs increased, the flexural capacity decreased, and the interface slippage increased. Also, for the same shear connection degree, the distribution of the studs in one row rather than two rows improved the flexural capacity while having no significant impact on the slippage behavior.

- The results of the numerical parametric study showed that best performances in terms of the ultimate capacity and tensile strains of the composite beams were obtained when both the depth of the steel beam the post-tensioning tendon eccentricity were increased.
• Numerical investigation of the shear stress range of the shear connectors showed that the effect of stud fracture on the shear stress range mainly manifested at the beam end where stud fracture occurred.

• When comparing the stress ranges calculated theoretically and those obtained from the FE model, a significant underestimation of the shear stress ranges experienced by the shear connectors was observed after fracture of 15% of the studs. This could be due to neglecting the steel-concrete interface slippage in the theoretical calculation of shear stress ranges of studs.

• Numerically investigating the effect of stud fracture on the effectiveness of post-tensioning showed that until the fracture of 15% of the studs, the strengthened sample had lower shear stress range, lower tensile and compressive strains, and higher residual capacity. After that, both the strengthened and non-strengthened samples exhibited similar responses to the failure of studs.

6.4 Recommendations for Future Work

This research covered experimental and numerical investigations of the fatigue and static behavior of steel-concrete composite beams. Although several areas were covered in this investigation, additional studies are necessary to expand the existent knowledge on the fatigue and static behavior of post-tensioned steel-concrete composite beams. Several recommendations for related future studies are listed below:
• Testing SCC girders under fatigue until failure is necessary to re-evaluate the current stress-fatigue life (S-N) curves that are available for the fatigue life of shear connectors in composite beams.

• In this research, the concrete flanges and shear connectors were visually inspected for fatigue cracks. It is recommended to utilize advanced technology such as Nondestructive Testing for crack detection in the shear connectors and the sides, top surfaces, and through the depth of concrete flanges.

• Experimental investigations to compare the effectiveness of different configurations of the post-tensioning tendons are of great importance. For example, externally post-tensioning using tendons with polygonal geometry with deviators within the support span.

• Experimentally applying different stress ranges to strengthened steel-concrete composite girders to evaluate the effect of the applied stress ranges on the fatigue behavior of SCC girders.

• Experimental and numerical investigations of continuous SCC beams strengthened using post-tensioning tendons.
Vita

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