

Faculdade de Engenharia da Universidade do Porto



Enhanced Fatigue and Seismic Performance of Metallic Structures Retrofitted with CFRP

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Abstract

This thesis focuses on the development of retrofit solutions and simplified design procedures for metallic structures using bonded carbon fibre reinforced polymers (CFRP). A comprehensive literature review on the behavior of CFRP/Steel adhesively bonded joints is presented. Most previous studies used brittle adhesives that may not provide the best performance for CFRP/Steel joints. The behavior of the CFRP/steel joints is studied extensively using experimental and numerical methods. A parametric study that includes several variants such as the adhesive type, CFRP type, CFRP length, adhesive thickness, and steel thickness, is conducted. The steel thickness influence significantly the behavior of CFRP/Steel joints, as thin steel plates tend to experience plastic deformation. The importance of using more representative specimens as generic tests is highlighted. Recent adhesives that combine high ductility and strength achieve very high performance. A high-strength cost-effective bi-adhesive CFRP/Steel joint is proposed for the construction industry. A literature review on the fatigue strengthening of metallic components using CFRP for the purpose of retrofitting old metallic bridges is presented. A statistical assessment and proposal of probabilistic S-N curves for CFRP retrofitted structural details is provided. A new approach to estimate the fatigue life of CFRP retrofitted structural details considering the fatigue degradation of CFRP is developed. The approach is validated with experimental data. An analytical fatigue crack growth model is developed based on experimental results from a database collected from the literature. The model is fitted with 70% of the data and validated with 30%. For the seismic retrofit of steel structures, the characterization of the behavior of steel beams subjected to large deformation cyclic flexural loading is provided, and deformation limits are proposed for the update of Eurocode 8 part 3. A CFRP retrofit solution is proposed to upgrade the deformation capacity of steel flexural members subjected to cyclic loading for the seismic retrofit of steel structures.

Resumo

Esta tese tem como foco o desenvolvimento de soluções de retrofit e procedimentos de projeto simplificados para estruturas metálicas usando polímeros reforçados com fibra de carbono (CFRP). Uma revisão abrangente da literatura sobre o comportamento de juntas adesivas de CFRP/aço é apresentada. A maioria dos estudos anteriores usou adesivos frágeis que podem não fornecer o melhor desempenho para juntas de CFRP/aço. O comportamento das juntas CFRP/aço é estudado extensivamente usando métodos experimentais e numéricos. É realizado um estudo paramétrico que inclui várias variantes, como o tipo de adesivo, tipo de CFRP, comprimento de CFRP, espessura do adesivo e espessura do aço. A espessura do aço influencia significativamente o comportamento das juntas CFRP/Aço, pois chapas finas de aço tendem a sofrer deformação plástica. Ressalta-se a importância do uso de corpos de prova mais representativos como testes genéricos. Adesivos recentes que combinam alta ductilidade e resistência alcançam um desempenho muito alto. Uma junta bi-adesiva CFRP/aço de alta resistência e custo-benefício é proposta para a indústria da construção. É apresentada uma revisão da literatura sobre o reforço à fadiga de componentes metálicos usando CFRP para fins de retrofit de pontes metálicas antigas. Uma avaliação estatística e proposta de curvas S-N probabilísticas para detalhes estruturais adaptados de CFRP é fornecida. Uma nova abordagem para estimar a vida em fadiga de detalhes estruturais adaptados de CFRP considerando a degradação por fadiga do CFRP é desenvolvida. A abordagem é validada com dados experimentais. Um modelo analítico de crescimento de trinca por fadiga é desenvolvido com base em resultados experimentais de um banco de dados coletado da literatura. O modelo é ajustado com 70% dos dados e validado com 30%. Para o retrofit sísmico de estruturas metálicas, é fornecida a caracterização do comportamento de vigas de aço submetidas a carregamentos de flexão cíclicos de grande deformação, e são propostos limites de deformação para a atualização do Eurocódigo 8 parte 3. Uma solução de retrofit de CFRP é proposta para atualizar a deformação capacidade de elementos de flexão de aço submetidos a carregamento cíclico para retrofit sísmico de estruturas de aço.

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CHAPTER 1: INTRODUCTION

1 Introduction

1.1 Motivation

Large number of the metallic infrastructures worldwide were built more than 50 years ago and are facing aging problems such as corrosion and fatigue. Throughout the service life of these structures, the nature and the level of loading have changed significantly. Old metallic infrastructures, such as bridges, high rise building, multi-storey parking and train stations, were design with conventional design codes (if any existed) that do not comply with the requirements of current design codes. Preserving these metallic structures in service and their resilience after extreme natural events such as earthquakes is a subject of paramount interest.

The US Department of Transportation revealed in 2008 that nearly 27% of the existing railway bridges in the US are structurally or functionally deficient [1]. Metallic bridges classified as deficient represent more than 43% of the total number. These bridges suffer from corrosion and fatigue. In Europe, more than 30% of the railway bridge stock has over 100 years old [2]. A large portion of these bridges is made of old mild steel or puddle iron, assembled with riveting technology. Theoretically, old riveted bridges have reached the end of their service life. The replacement of old metallic bridges is economically unfeasible. Therefore, there is an urgent need for the development of retrofitting solutions which are effective and significantly less costly.

Many important steel buildings, such high-rise building, play an important socio-economic role in metropolitan regions. The resilience of these buildings when subjected to extreme loadings, such as earthquakes, wind, and tsunamis, is of high importance. Large portion of these steel buildings were erected between 1950's and 1990's according to conventional design codes and standards that do not meet the requirement of current design codes [3]. Steel-framed buildings were traditionally believed to have a very ductile behavior and would withstand very large deformation and dissipate energy when subjected to extreme loadings such as Earthquakes. However, post-earthquake reconnaissance in Kobe Japan (1995) and Northridge USA (1994) showed that steel buildings suffered from low cycle fatigue cracks and achieved very low ductility. Besides, state of the art research [4], [5] showed that the

deformation capacity of steel structural members subjected to cyclic loading is very low compared to the monotonic behavior traditionally assumed for design. Eventually code-based seismic design requirements for structural members are going to be significantly penalized [6] which raised lot of discussion in the literature [7], [8]. The new design codes requirements cause rejection of buildings that were assumed seismically resistant few years ago and downgrade the performance classification of existing steel buildings. Therefore, simple and effective retrofit solutions are strongly needed to upgrade seismic performance of existing buildings.

1.2 FRP strengthening solution

The development of reinforcement solutions for the rehabilitation of metallic structures is a subject of paramount interest. Fibre Reinforced Polymers (FRP) are increasingly popular among several engineering disciplines. Since three decades ago, bonded FRP repairs are well established in aerospace and aeronautical engineering for lightweight metallic structures [9], [10]. The use of a composite patch repair brings through the following advantages:

- Increase strength of the structural component considerably.
- Reduces the stress concentration and promote a more uniform stress distribution.
- Does not alter the mechanical and fatigue properties of the parent material as in the case of drilling or welding.
- Due to the lightweight of the laminates, the transport becomes less costly, and no additional loading is added to the structure.
- More importantly, the application of this technology in practice is more straightforward than any existing methods, which can reduce the labour cost significantly.

Owing to the superior properties of this technology listed above, the use of bonded FRP patch can be a very practical solution to reinforce metallic structural components. Recently, lot of research attention was given for the application of FRP patch to extend the fatigue life in metallic bridges [11]–[15], to enhance the buckling resistance of steel members [16]–[21]. However, the application of this technology in practice remains very limited for metallic structures. This is mainly due to the absence of

design code worldwide. Currently only two design guidelines (i.e. CIRIA 595 [22], CNR-DT2005 [23] and EUROCOMP [24]) which acknowledge their limitations especially when cyclic or fatigue loading is present. The situation is completely different for concrete structures where the use FRP reinforcements is well established and comprehensive internationally recognized design codes are available (ACI 440, [25]).

In order to implement this reinforcement technology in practice for metallic structures, more experimental research that highlight its effectiveness is needed. The behavior of the adhesively bonded composite patch is very complex that involves material nonlinearity and fracture mechanics. For civil engineering practice, simplified design procedures need to be developed and implemented in design guidelines.

1.3 Aim and objectives:

The aim of this thesis is to mainly to propose effective retrofit solutions for metallic components using FRP and develop simplified design procedures that can be used by engineers in practice.

The focus of this research is on the application of adhesively bonded Carbon Fibre Reinforced Polymers (CFRP) to repair or upgrade metallic structural components. Therefore, a large experimental campaign was first conducted to characterise the behavior of CFRP/steel adhesively bonded joints and identify which type of adhesive, CFRP, and method of application are best suited for the repair metallic structures. Then, investigation on the repair/upgrade metallic structural components with CFRP were carried out for mainly two different contexts:

- Fatigue strengthening of bridge components:
- Seismic rehabilitation of metallic buildings:

The thesis is divided mainly in three parts:

a) PART I: Characterization of CFRP/Steel adhesive joints

The objective of this study is to characterize the behavior of the adhesives and steel/CFRP joints experimentally and numerically and identify what is the most suitable solution to repair metallic

structures. Firstly, a state-of-the-art review is presented. Then, experimental and numerical parametric study is conducted to investigate the effect of several variants such as: the effect of different adhesive types, different geometrical configurations, CFRP types, and surface treatments. Finally, a cost-effective adhesive joint based on mixed adhesive bond lines was proposed for the construction industry.

b) Part II: Application of CFRP to extend the fatigue life of metallic components

As mentioned previously, the fatigue maintenance of aging metallic bridges is a concern worldwide. Since lot of experimental research was published in the literature on this topic. This part of the thesis is mainly focused on the derivation of simplified fatigue design methods for metallic structural details repaired with CFRP based on a database of the experimental data collected from the literature. Therefore, this part of the thesis includes 3 chapters which consists of:

- A statistical assessment of fatigue data for CFRP retrofitted non-cracked specimens.
- A proposal of a new approach to estimate the fatigue life of CFRP retrofitted non-cracked structural details.
- A new fatigue crack propagation model for CFRP reinforced cracked structural details.

PART III: Application of CFRP to extend the ductility of flexural members for the seismic design of steel structures

The focus of this study is to characterize the cyclic behavior steel beams subjected to large cyclic flexural loading and propose a reinforcement strategy. For this purpose, this study was carried out in two steps, as following:

- A characterization of the deformation capacity of steel beams with European profiles subjected to cyclic loading is conducted and empirical equations to estimate the plastic rotation capacity of steel beams are proposed for the update of Eurocode 8 part 3.
- A retrofit solution using bonded CFRP is proposed to increase the ductility of steel beams subjected to large reversal cyclic loading. A high-fidelity finite element analysis

is conducted for this purpose and the comparison between different adhesive type is provided.

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**Part I: On the characterization of CFRP/Steel
adhesively bonded joints**

Chapter II: State of the art review on the behavior of CFRP/Steel adhesively bonded joints

2 State of the art review on the behavior of CFRP/Steel adhesively bonded joints

Abstract:

This chapter presents a state of the art review on the behavior of CFRP/Steel bonded joint used in the construction industry-related research. A comparison between the behavior of CFRP/concrete joints and CFRP/steel joints is provided. The stress distribution and the failure modes observed in CFRP/Steel adhesive joints are discussed. A critical discussion and recommendations on the experimental methods, the adhesives used, and the analytical proposed in the literature, are provided. Then finally the numerical approach commonly used to simulate the damage initiation and propagation in the adhesive layer is presented.

2.1 Introduction

The use of bonded Carbon Fiber Reinforced Polymers (CFRP) for strengthening existing structures is widely used in several engineering disciplines. It played an important role in military engineering during World War II and, nowadays, is extensively used in several engineering fields, such as aerospace, aeronautics, marine, and automotive. In civil engineering, the application of CFRP is becoming very common to retrofit reinforced concrete structural components. However, for steel structures, only limited case studies have been implemented worldwide. In 2011, Harries [1] presented several case studies of existing metallic structures in the U.K that have been reinforced with CFRP laminates. Most of them were old metallic bridges and train stations built more than a century ago with cast iron and old mild steel. Figure 2-1 illustrates a field repair intervention of riveted plate girders of an old bridge located in Acton, London, where the live load stresses were successfully reduced by 24% [1].

The application of bonded CFRP to steel structures is a niche field that deserves significant research attention. The main challenge of this technology remains in the characterization of the adhesive joint behavior, which can differ significantly based on the adhesive type, adherends stiffness, geometry of the

joint, surface preparation, etc. For civil engineering applications, there are no standardized experimental testing techniques for the characterization of the adhesive joint behavior. Therefore, the testing protocols used in the literature can differ significantly.

This chapter provides a state-of-the-art review that covers different aspects related to the characterization of adhesively bonded CFRP/Steel joints. Firstly, a comparison between the behavior of adhesively bonded repairs for concrete and steel structures. Secondly, the fundamental behavior of adhesive joints is presented which includes the stress state distribution and the failure modes. Then a critical review and recommendations on the experimental methods are provided, by showing the shortcomings of the current specimens' configurations and highlighting the need of using more representative specimens. Subsequently, the different analytical models based on the stress approach or fracture mechanics approach provided in the literature to estimate the bond strength are presented with careful consideration of their applicability to steel construction structures for reinforced configurations. Then finally the numerical approach commonly used to simulate the damage initiation and propagation in the adhesive layer is presented.



Figure 2-1: Application of CFRP to Acton Bridges, London (Harries, [1]).

2.2 Comparison between the CFRP strengthening of steel and concrete structures

In the last three decades, extensive research has been devoted to develop design guidelines for the application of bonded CFRP strengthening for concrete structures. Nowadays, design procedures and modeling methods are well established in design codes ready to be used by practitioners and design engineers for concrete structures (ACI 440, [2]). However, the application of CFRP to steel structures is not very popular as it is in the case of concrete structures. The reason for this can be attributed to the complex behavior of the adhesive CFRP/steel joint that needs to be considered to derive design procedures.

Although the characterization of the behavior of a concrete/CFRP adhesive joint can be very complex too, the derivation of design rules can be very simple. The rationale behind this reason is that structural adhesives available in the market are much tougher than cementitious materials such as concrete. The weakest part of the CFRP/adhesive/concrete joint is in the parent material (i.e. concrete). Therefore, the damage is unlikely to start in the adhesive or the CFRP laminates. Adhesive debonding may occur after reaching the ultimate state of the concrete structural element. Figure 2-2 illustrates the classical failure modes of a reinforced concrete beam strengthened with FRP, where all modes involve prior concrete cracking. Concrete strength is one of the main parameters affecting the behavior of the CFRP/adhesive/concrete joint [3]. Usually, design engineers and researchers with a civil engineering background have good knowledge of the behavior of reinforced concrete elements. Thus, design methods and procedures could be proposed and applied naturally.

Conversely, in steel structures reinforced with CFRP, the weakest link in Steel/adhesive/CFRP joints can be in the adhesive, CFRP, and/or steel. To characterize the behavior of CFRP/steel adhesive joints, fracture mechanics tests and advanced finite element modeling are inevitable. Currently, there is a wide range of adhesives that differ in strength, ductility, and toughness. The use of different adhesives can change the behavior significantly. Even though, the yield strength of the strongest adhesive (i.e. 60MPa) is way lower than the yield strength of steel, however, when loaded in shear over a large area, extremely tough adhesives are sufficiently strong to cause yielding in steel [4]. Damage in CFRP is also commonly reported in the

literature for CFRP/steel adhesive joints [5]. Due to the complex behavior of steel/CFRP adhesive joints, it is much more challenging to derive design procedures. Currently, design guidelines for steel structures such CIRIA 595 [6] and CNR-DT2005 [7] are very descriptive and acknowledge their limitations due to the lack of experimental data, especially when cyclic or fatigue loading is present. The partial safety factors are as high as the value of 3. With the available knowledge presented in design codes, it is almost impossible for practitioners to provide a retrofit design of steel structures with CFRP. Yet, all of the case studies implemented in existing metallic structures required stakeholders from academia or the research industry to undertake experimental testing and in-situ monitoring.

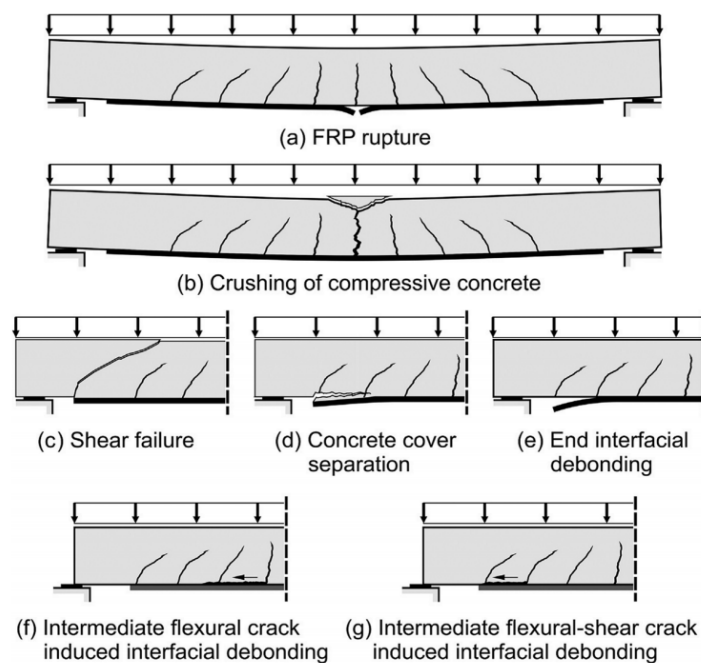


Figure 2-2: Classical mode of failures for concrete-CFRP [8].

2.3 Stress distribution in CFRP/Steel adhesive joint

Mapping the stress and strain distribution inside the adhesive joint is a very difficult task, and can differ significantly depending on the geometry, stiffness of the adherends, and the thickness of the adhesive layer. The stress state inside the adhesive is decomposed into a shear stress component and a normal stress component.

Figure 2-3 illustrates the stress state inside the adhesive for a single lap joint. Figure 2-3 (a) shows that the force transfer is achieved by shear transfer in the adhesive layer, where the stress distribution reduces through the length until the full composite action is ensured. Exceeding this length would not increase the axial force transferred to the CFRP laminate. Figure 2-3 (b) illustrates that the eccentricity between the steel plate and the resins creates a secondary bending effect that is counteracted by normal stress to ensure equilibrium. This normal stress causes the peeling of the CFRP plates, which invokes premature damage in the adhesive joint. According to Deng and Lee [9], at the bonding joints, crack initiates and grows in mode I due to peeling stress, earlier than in mode II. The most common solution used to reduce premature failure by peeling was proposed by Vin son & Sierakowski (1987) [10], where the CFRP plates must be beveled to a 45° angle at the ends of the plate. The effectiveness of this method is reported in the experimental program conducted by Miller [11]. The European code for composite structures (EUROCOMP) [12] (p.162) suggests also other techniques such as: increasing the adherend stiffness without increasing its thickness, increasing the lap length, tapering the ends of the adherends, and using adhesive fillets.

2.4 Failure modes in CFRP/steel adhesive joint

Identifying failure modes is critical to understand the behavior of the adhesive joint and derive design rules. The possible modes of failure that can occur in the CFRP/Steel joint are shown in Figure 2-4, which consists of : (a) adhesive failure Steel-adhesive interface; (b) adhesive failure FRP-adhesive interface; (c) cohesive failure in the adhesive failure adhesive layer; (d) CFRP rupture; (e) CFRP delamination; and (f) steel yielding.

The failure mode can be influenced by several factors such as:

- **Surface treatment:** A poor surface treatment of the adherends results in premature adhesive failure. A proper surface treatment (e.g. sandblasting for steel substrates and sandpaper polishing

for CFRP) provides a rough surface that favors mechanical interlocking between the adherends and the adhesive which increases the adhesion strength.

- **CFRP elastic modulus:** The CFRP with a very high elastic modulus (i.e. $E > 300 \text{ GPa}$) usually have low deformation capacity at failure and CFRP rupture becomes the dominant failure mode as observed in most experimental results [13].
- **Adhesive type:** The strength, ductility, and toughness of the adhesive influence significantly the failure mode in CFRP/Steel adhesive joint. Brittle adhesives such as the ones used in the construction industry have very low toughness leading to cohesive failure, as reported in the literature [14]. Extremely tough adhesives that combine high strength and ductility may yield the steel in shear and CFRP delamination may govern the behavior.
- **Adhesive thickness:** The influence of the adhesive thickness on the behavior of CFRP/steel adhesive joints is a complex issue. The adhesive thickness influences the stress state in the adhesive layer. For instance, thicker thickness increases the peeling stress discussed as discussed in the previously. Thicker adhesive bond lines are more likely to have air bubbles.
- **Steel yield strength:** adherend with low yield strength may influence the behavior significantly if yielding occurs in the CFRP/steel adhesive joint.

2.5 Critical review Experimental testing techniques

Different configurations and testing methods exist in adhesion technology. They usually depend on the purpose and adhesive material characteristics. Detailed information on the pros and cons of the different specimen configurations can be found in adhesion technology textbooks, such as ref. [16], [17]. Usually, they distinguish two categories: strength failure tests and fracture mechanics tests. Fracture mechanics testing methods are well developed in adhesion technology, the reader can refer to specialized textbooks such as ref. [17]. The double Cantilever Beam (DCB) test and End notched Flexure (ENF) tests are the most common tests used to evaluate the fracture energy release rate for fracture modes I and II, respectively. However, for civil engineering applications, these types of testing methods are quite rare. Strength failure

tests such as the single shear lap joint test (see Figure 2-5) and the double strap joint test (see Figure 2-6) are very common. Most the analytical and design formulations such as the Hart-Smith model [18] or Xia and Teng model [19] are derived using those specimens. The single lap shear joint consists of a rigid steel block bonded with CFRP which is in turn pulled in the horizontal direction. The bonded region is usually very long which exceeds 300mm. The double strap joint consists of two steel bars joint together with bonded CFRP in both sides.

For the development of design procedures, generic joints representing the behavior of the CFRP patch repair in a real situation should be used. For the fatigue crack repair in aerospace applications, Chalkey et al. [20] proposed two generic specimens to be used to represent the behavior of a CFRP patch crack repair, as shown in Figure 2-7. The specimens consist of a double lap joint to represent the behavior in the disbond-tolerant zone and the double skin doubler to represent the behavior in the safe life zone, see Figure 2-8.

Haghani and Al-Emrani [21] compared the stress state in (a) steel plate reinforced with CFRP on both sides (see Figure 2-9); (b) single shear lap joint; (c) double strap joint; (d) and an I shaped steel beam reinforced with CFRP on the lower flange subjected to 3 point bending tests. The results showed that the stress state inside the adhesive layer of a reinforced plate is very similar to the reinforced beam. However, the single lap and the double lap joints showed a completely different stress state than the beam. Matta et al. [22] also compared double lap joint and double reinforced specimen, it was shown that bare steel plates reinforced with CFRP in both sides and double strap joints have different strain distribution and failure modes. Steel plates reinforced with CFRP in both sides failed under adhesive failure, and the double strap joint failed under cohesive mode, a similar observation was also reported in ref. [21].

Based on extensive literature reviewed, all of the tests conducted on reinforced steel beams or steel bars reinforced with CFRP on both sides reported adhesive failure mode, one can refer to ref. [9], [21]–[28]. However, most of the design rules are derived based on cohesive failure observed in single lap joint or double strap joint tests.

The behavior between lap joints (single lap and double strap joints) and CFRP reinforced bars (without a notch) is significantly different due to the stress transfer. In lap joints, the stress is transferred from the CFRP to steel in the middle of the joint, whereas in reinforced bars from the steel to the CFRP at the end of the overlap. This influences the stress state and the failure modes significantly in the adhesive joint. Based on numerical evidence, double strap joints are representative of a CFRP patch covering a cracked area in the middle or large notch that causes most of the deformation concentrated in the middle of the patch. Whereas for steel plates without a notch or a crack such as in steel beams, the behavior is similar to a steel bar reinforced with CFRP, as shown in see Figure 2-9.

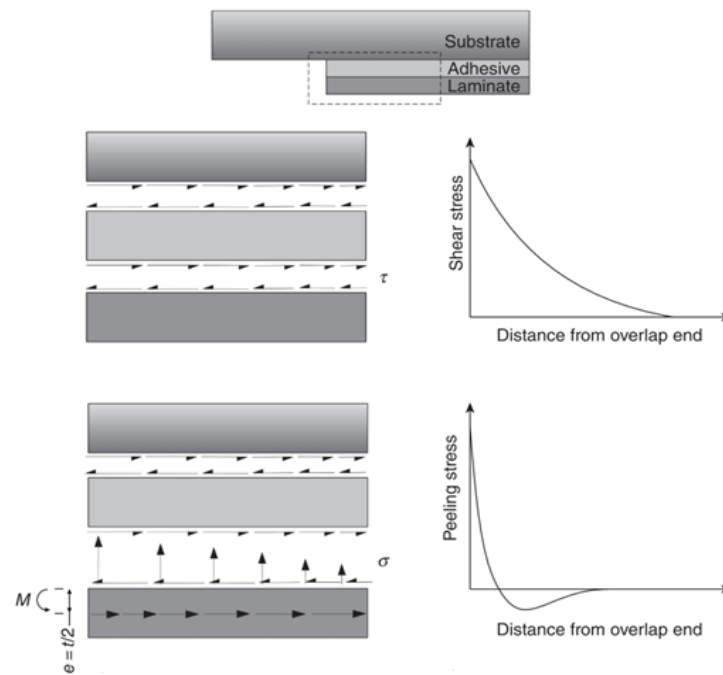


Figure 2-3: force transfer mechanism between Steel and CFRP laminates [15].

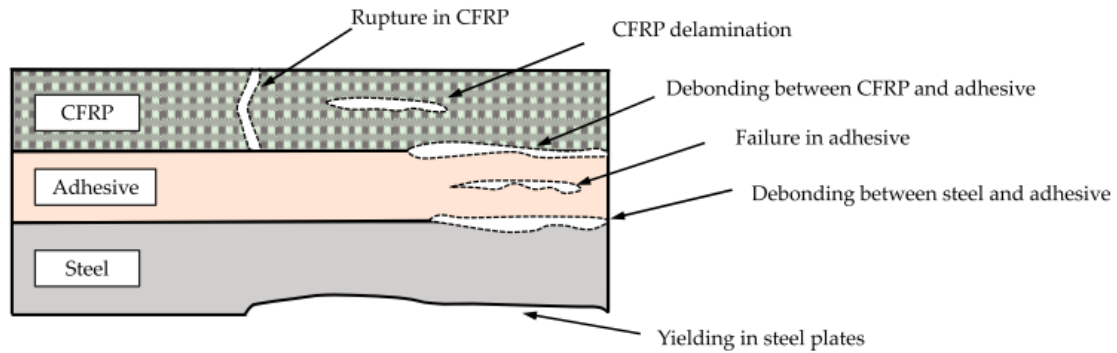


Figure 2-4: schematic view of a possible mode of failures.

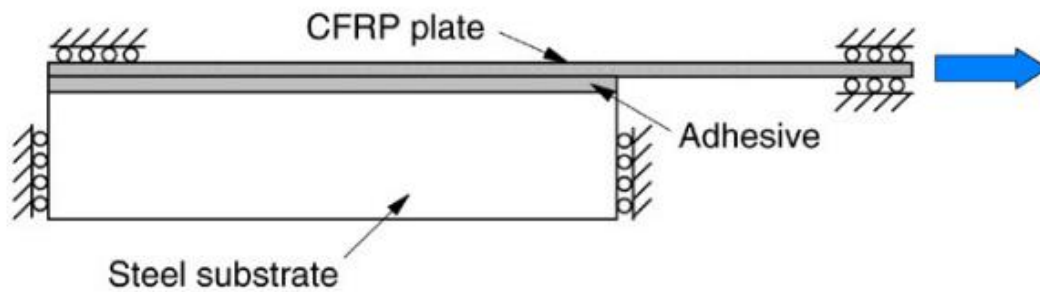


Figure 2-5: Single lap shear joint [13].

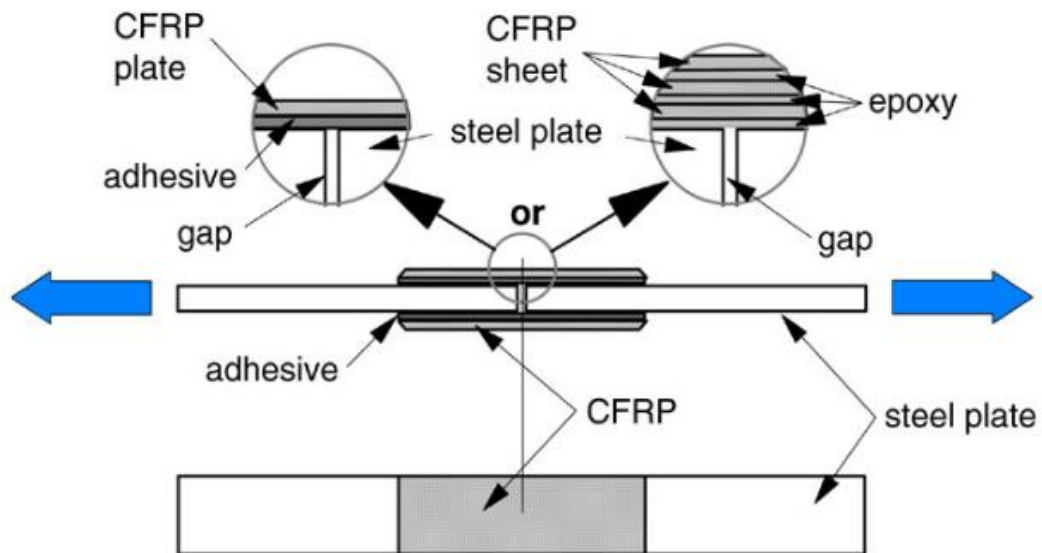


Figure 2-6: Double strap joint [13].

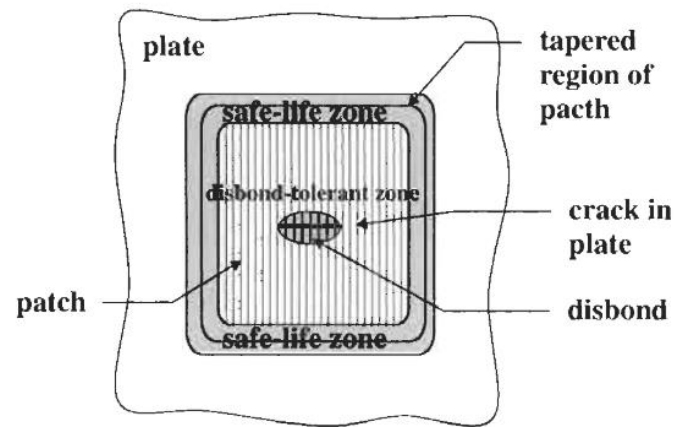


Figure 2-7: CFRP patch crack repair [20].

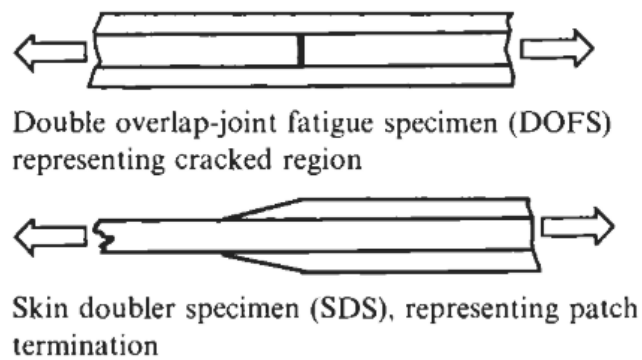


Figure 2-8: Generic specimens to assess the behavior in the CFRP patch repair [20].

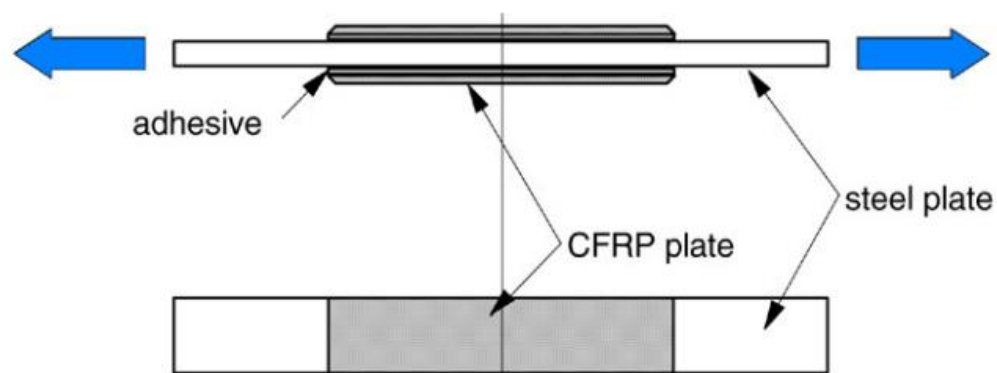


Figure 2-9: Steel bar reinforced with CFRP on both sides.

2.6 Surface treatment of the adherends

Surface treatment refers to the preparation of the adherends surfaces for bonding. Usually, it is done by polishing the surfaces and cleaning using chemical solvents such as acetone to degrease the surface. The surface treatment can have a strong influence on the failure mode. Poor surface treatment usually causes a premature adhesive failure mode. EUROCOMP [12] (p.158) states that the adhesion mode of failure that occurs at the adhesive-adherend interface, occurs due to material mismatch or inadequate surface treatment, and it should be avoided. According to Teng et al [9], cohesive failure is the preferred mode since it depends only on the properties of the adhesive material, thus it would be much simpler to establish a design theory. When the surface is adequately treated, the adhesion strength can be greater than the cohesion strength, thus it allows efficient use of the full strength of the adhesive until failing in cohesive mode. More information about surface treatment can be found in Teng et al. [29], Fernando et al. [30].

Premature adhesive failure depends on the surface treatment of the adherends, and it is very difficult to control it on site. Several methods exist for surface treatment including:

- Roughening the surface and removing the weak, chemically inactive oxide layer.
- Grit blasting with a finer grit (e.g. 0.25mm).
- Sandpapers, wire brushes, abrasive pads and wheels, needle guns, etc.
- Degreasing – removing contaminants like oil, grease, and water.
- Solvent cleaning.
- Brushing, ultrasonic degreasing, and vapor degreasing.
- By dry wipe or vacuum head.

Fernando et al [30] tested two series of butt joint tensile tests and single lap shear tests. Four adhesives and five different surface preparation methods were included in the experimental program. A summary of the results obtained for shear lap tests is provided in Table 2-1.

Table 2-1: Single lap shear tests failure modes for different adhesives and surface preparation [30].

Ultimate load KN (failure mode)						
Adhesive	Specimen	Solvent-cleaned	Hand-ground	0.5mm grit-blasted	0.25mm grit-blasted	0.125mm grit-blasted
SIKA 30	I	2.91(a)	3.31(a)	4.08(c)	4.97(c)	4.43(c)
	II	2.81 (d)	3.87(a)	4.43(c)	4.32(c)	4.44(c)
	III	2.38(d)	3.39(a)	5.08(c)	4.76(c)	4.81(c)
SIKA330	I	3.66(d)	4.72(a)	5.48(a)	6.09(a)	5.74(a)
	II	3.88(d)	4.73(a)	5.30(a)	5.46(a)	5.60(a)
	III	3.45(d)	4.96(a)	5.79(a)	5.47(a)	5.70(a)
Araldite 2015	I	2.86(a)	2.73(a)	3.54(a)	4.37(a)	4.09(a)
	II	1.88(d)	2.48(a)	3.77(a)	4.04(a)	3.82(a)
	III	2.85(a)	2.89(a)	3.50(a)	4.04(a)	4.13(a)
Araldite 420	I	3.67(d)	5.23(a)	5.42(a)	5.70(c)	5.62(c)
	II	3.04(a)	5.30(a)	5.40(c)	5.15(c)	6.65(c)
	III	3.56(d)	5.11(a)	5.45(a)	6.01(c)	5.39(c)

(a) Combined failure mode between cohesive and adhesive.

(c) Cohesive failure mode.

(d) Adhesive failure mode.

In conclusion, Fernando et al. [30] recommended for practical applications to achieve a surface topography of $1.49 > D_f > 1.47$ that can be achieved by sand blasting using grit size between 0.125 and 0.25 Doroudi et al. [31] used the aforementioned recommendations and shown to achieve cohesive failure when using a double-strap joint.

2.7 The choice of adhesive

The choice of the adhesive for a particular application is the most important part to achieve successful bonding. Usually, there are a variety of adhesives that can be used. For civil engineering applications, epoxy-based structural adhesives are adopted due to their high strength and their capability to achieve high bonding strength with construction materials such as concrete. Different epoxy-based structural adhesives can be found in the market. They can differ greatly with respect to their use and mechanical properties. They can be classified based on their stiffness, strength, and ductility. Figure 2-10 shows three different structural adhesives classified based on their mechanical properties, as follows:

- Rigid adhesives: also called brittle adhesives are usually distinguished by their high elastic modulus (4.5GPa-13GPa) and high strength (>30MPa).

- Ductile adhesives: have lower elastic modulus (1GPa-2GPa), moderate strength (~ 20 GPa), and high ductility ($>10\%$).
- Extremely tough adhesives: combine high strength and ductility.

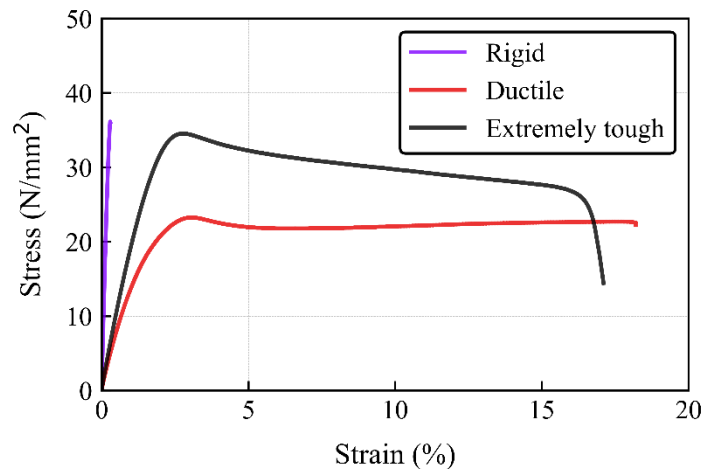


Figure 2-10: structural adhesive types based on their mechanical performance.

In the construction industry, only rigid adhesives can be found in the market. Most of the previous research studies used rigid adhesives which were developed for concrete structures. The choice of the adhesive is usually due to the recommendation of the commercial provider. Although these brittle adhesives can bond well to steel substrates, they may not provide the best performance. The mechanical properties of the adhesive have a significant influence on the performance of the steel/CFRP adhesive joints. Recent tough adhesives with high strength and ductility are more appropriate for bonding metallic substrates [32]. Very recently, SIKA leader in the construction market [33] developed a moderately tough adhesive (i.e. Sikadure 370) for the repair of steel bridges affirming that the highly brittle 2C-epoxy adhesives used in construction practice are more suitable for concrete and other cementitious systems but for more ductile substrates such as steel toughened ductile adhesives should be used. Kasper et al. [34] characterized the behavior of the Sikadure 370 and compared it with the most common brittle adhesive Sikadure 30. The results revealed

that the tough adhesive is more suitable for the strengthening of steel bridges due to its high strength and durability.

2.8 Evaluation of bond strength under static loading

Generally, there are two theoretical approaches and one empirical approach commonly used for the design of the adhesive joint. The theoretical approaches are the stress-based approach and the linear fracture mechanics approach. The empirical approach is called the bond-slip approach. This method makes indirect use of the fracture mechanic approach by assuming the fracture energy release rate equals the area under the shear-bond slip curve obtained using a failure strength test.

Unfortunately, despite the considerable efforts, most of the design rules and analytical approaches provided in the technical literature, are based on the assumption of cohesive failure. Whereas it was shown previously that adhesive failure is the expected mode of failure for CFRP reinforcement in steel construction. This was confirmed in the paper of Haghani [35] who demonstrated that the existing design philosophy based on cohesive failure would not produce accurate results for beams, and it was suggested that new design methods based on adhesive failure are needed. To the knowledge of the author, there are very limited works (e.g. ref. [36]) that assumed adhesive failure using specimens that are more representative of civil engineering applications.

This section will summarize the most relevant studies conducted on the evaluation of the bond strength between CFRP and steel. Even though cohesive failure is assumed to be not a representative mode of failure, the existing bond strength models based on cohesive failure are presented. Then, it will be followed by a presentation on the few studies conducted to estimate the bond strength under adhesive failure.

2.8.1 Models based on cohesive failure

2.8.1.1 The stress-based approach

The stress and strain methods have been extensively studied in the literature due to their simplicity. Usually, researchers in engineering fields are more familiar with stress and strains than fracture mechanics. The

philosophy behind this approach is to estimate the stress demand and compare it with the characteristic strength of the material or the joint. Different methods have been suggested to estimate the stress demands and various assumptions were recommended to evaluate the bond strength, more information can be found in ref. [21]. The common criteria used to estimate the demands are listed in the following:

- (a) *The average stress criterion*: assumes a uniform stress distribution inside the adhesive joint, for example for a single lap joint, the average stress can be assumed equal applied force divided by the area of the adhesive layer. However, the stress and strain inside the adhesive joint are not constant except for very short joint with very stiff adherends, which is not the case for CFRP strengthened steel. Also, the initiation of failure occurs usually in the bond end where there is the zone of high-stress concentration. This method ignores completely this effect which can lead to an underestimation of the stress demands.
- (b) *Maximum stress criterion*: assumes that failure occurs whenever stress or strain reaches the critical capacity at any point in the adhesive joint. Some suggest to verify the maximum shear or peeling stress separately without considering the mixed-mode effect which is highly present in the adhesive joint between CFRP and steel. Another methodology suggests considering the maximum principal stress in the adhesive joint. This methodology was adopted in conjunction with the Finite element method by some researchers such as Harris and Adams [37]. However, the evaluation of the maximum stress in the adhesive joint using FE analysis is mesh-dependent and very sensitive to fillet geometry. Therefore there is a range in the neighbourhood of stress singularities that cannot be estimated correctly.
- (c) *Maximum principal stress in conjunction with representative specimen*: The main shortcoming of the aforementioned methods is that they are checked with the strength of the bulk adhesive material. Where in reality the geometry and the stiffness of the adherends influence significantly the behavior of the adhesive joint. Therefore, it is more appropriate to consider the representative adhesive joint. This method is adopted by design guidelines such as CIRIA [6] using equation (1) for the design verification and assuming shear lap and double strap joint as representative joints. This method is presented to show

what is provided in the design codes. The peeling (σ) and shear (τ) stress can be estimated using closed-form analytical solutions provided in Appendix 5 of CIRIA [6] or comprehensive numerical methods.

$$\sigma_1 = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \leq \bar{\sigma} \quad (1)$$

The characteristic strength $\bar{\sigma}$ corresponds to the peak stress at failure in the lap shear test. Note that the shear lap test provides ultimate applied force at failure divided by the total area provides the average shear stress. Therefore, in Appendix 6 of CIRIA [6] design code, a back analysis approach is provided to estimate the peak shear stress. Equation (1) is factorized by very high partial factors due to the high statistical uncertainty related to the applied loads, design, and material property.

In the case of a double strap joint, CIRIA [6] adopted the HART-SMITH model [18] which provides an analytical formulation to evaluate the maximum failure load on a double lap joint. The HART-SMITH model for double strap joint is described mathematically as following:

$$P = \min \begin{cases} P_i = \sqrt{2 \cdot \tau_p \cdot t_a \cdot \left(\frac{1}{2} \cdot \gamma_e + \gamma_p\right) \cdot 2 \cdot E_i \cdot t_i \cdot \left(1 + \frac{E_i \cdot t_i}{2 \cdot E_o \cdot t_o}\right)} \\ P_o = \sqrt{2 \cdot \tau_p \cdot t_a \cdot \left(\frac{1}{2} \cdot \gamma_e + \gamma_p\right) \cdot 4 \cdot E_o \cdot t_o \cdot \left(1 + \frac{E_o \cdot t_o}{2 \cdot E_i \cdot t_i}\right)} \end{cases} \quad (2)$$

Where γ_e and γ_p are the elastic and plastic adhesive shear strains, respectively.

P is the ultimate load-carrying capacity per unit width.

The effective bond length can be calculated as follows:

$$l_e = \frac{\sigma_{ult} t_a}{\tau_p} + \frac{2}{\lambda} \quad (3)$$

$$\lambda = \sqrt{\frac{G_a}{t_a} \left(\frac{1}{E_o t_o} + \frac{2}{E_i t_i} \right)} \quad (4)$$

σ_{ult} is the ultimate tensile strength inside the adherent, τ_p is the adhesive shear strength, G_a is the adhesive shear modulus, t_a is the adhesive thickness, E Young's modulus of the adherent, and t is the thickness of the adherent. The subscripts o and I represent outside and inside adherent, respectively.

2.8.1.2 Fracture mechanics approach

The most common method is the linear fracture mechanics approach that makes use of the strain energy release rate (G) that is associated with the propagation of a crack in the adhesive layer. It usually involves three basic steps which consist of calculation of the energy release rate for a given adhesive joint, estimation of the critical values of the energy release rate for the material, and finally, applying a failure criterion as in equation (5). CIRIA [6] design guidelines suggest considering only Mode I (peeling) as the dominant failure mode since the critical energy rate in mode II is much higher than in mode I. Therefore, the design rule consists of satisfying equation (5).

$$G_I \leq \bar{G}_{IC} \quad (5)$$

Where $\bar{G}_{IC}$ the critical energy release rate also called toughness of the adhesive joint can be determined using fracture mechanic tests. As an illustrative example, for a beam G_I the energy release rate is defined by:

$$G_I = \frac{1}{b_a} (u_1 - u_2) \quad (6)$$

Where u_1 and u_2 are the strain energy per unit length for strengthened and unstrengthened beam section respectively, and b_a is the width of the adhesive layer, more information can be found in CIRIA [6] appendix 7.

2.1.1.1 Bond slip approach

The bond-slip behavior represents the relationship between the interfacial shear stress and the relative slip between the CFRP and the steel substrate. The bond-slip approach is extensively used in the literature. It makes indirect use of fracture mechanics by estimating the fracture energy at failure for a strength failure test which is assumed to be equal to the area under the shear-bond slip curve. Xia and Teng [19] were the

first to adopt this model for steel specimens bonded with CFRP, which was based on the work of Yuan et al[38] model that was in turn based on the fracture mechanics model proposed by Täljsten [39] for FRP-to concrete using single lap steel-CFRP joint test.

The main assumptions of the bond-slip model are:

- Only fracture mode II is considered.
- Both adherends are assumed to be subject to uniformly distributed axial stresses only.
- Any bending effects were neglected.
- The materials are homogenous, isotropic, and linear elastic.
- The thickness of the adherends and the adhesive are constant throughout the bond line.
- The width of the steel plate is constant throughout the bond line.

Remark: the assumption of only shear (mode II) is not realistic for CFRP applied to steel. It was taken from the application to concrete where the peeling stress can be much lower. Besides, it was shown in previous sections that mode (I) is very important and for example CIRIA595 guideline [6] bases all the design methodology on mode I when using fracture mechanics.

Figure 2-11 (a) and (b) illustrate the bond-slip models for linear adhesive and nonlinear adhesive, respectively. As can be seen, rigid adhesives tend to have a more triangular curve and ductile adhesives have a trapezoidal shape. The mathematical representation of the triangular bond-slip model is as follows:

$$\tau = \begin{cases} \tau_{max} \frac{s}{s_1} & s \leq s_1 \\ \tau_{max} \frac{s_f - s}{s_f - s_1} & s_1 \leq s \leq s_f \\ 0 & s \geq s_f \end{cases} \quad (7)$$

And for the trapezoidal model:

$$\tau = \begin{cases} \tau_{max} \frac{s}{s_1} & s \leq s_1 \\ \tau_{max} & s_1 \leq s \leq s_2 \\ \tau_{max} \frac{s_f - s}{s_f - s_2} & s_2 \leq s \leq s_f \\ 0 & s \geq s_f \end{cases} \quad (8)$$

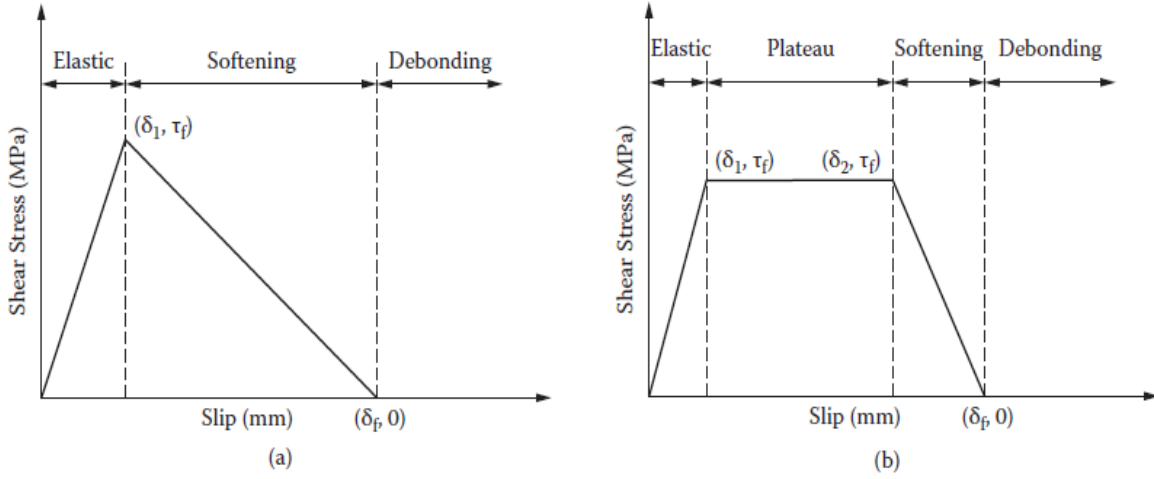


Figure 2-11: Bond slip model types: (a) Triangular bond-slip model; (b) Trapezoidal bond-slip model.

Different models and parameters have been proposed for the triangular and trapezoidal bond-slip models, as shown in Table 2-2 and Table 2-3, respectively.

Table 2-2: Triangular bond-slip models.

Bond slip model	τ_f	δ_1	δ_f
Xia and Teng[19]	$\tau_f = 0.8f_{t,a}$	And $\delta_1 = \tau_f/K_a$ $K_a = G_f/t_a$	$\tau_f \delta_1 = 62(G_f/t_a)^{0.56} t_{adh}^{0.27}$ And $G_f = \frac{\tau_f \times \delta_f}{2}$
Fawzia et al [40]	$\tau_f = f_{t,a}$	$\delta_1 = t_a/10$	If $0.1 \leq t_a \leq 0.5mm$ $\delta_f = t_a/4$ If $0.1 \leq t_a \leq 1mm$ $\delta_f = 0.125 + \frac{t_a - 0.5}{10}$
Fernando [41]	$\tau_f = 0.9 f_{t,a}$	$\delta_1 = 0.3 \left(\frac{t_a}{G_a} \right)^{0.65} f_{t,a}$	$\delta_f = \frac{2G_f}{f_{t,a}}$
Wang andWu [42]	$\tau_f = 0.9f_{t,a}$	$\delta_1 = \frac{2.6f_{t,a}}{G_f} t_a^{0.34}$	$\delta_f = 540 \frac{t_a^{0.4} R^{1.7}}{f_{t,a}}$

Table 2-3: Trapezoidal bond-slip models.

Bond slip model	τ_f	δ_1	δ_2	δ_f
Wang and Wu [42]	$\tau_f = 0.9f_{t,a}$	$\delta_1 = \frac{2.6f_{t,a}}{G_f} t_a^{0.34}$	$\delta_2 = 180 \frac{t_a^{0.4} R^{1.7}}{f_{t,a}} + 0.85 \frac{t_a^{0.34} f_{t,a}}{G_a}$	$\delta_f = 360 \frac{t_a^{0.4} R^{1.7}}{f_{t,a}} + 1.7 \frac{t_a^{0.34} f_{t,a}}{G_a}$
Dehghani [43]	$\tau_f = 0.8f_{t,a}$	$\delta_1 = \frac{\tau_f}{G_a} t_a$	$\delta_2 = \frac{1}{3} \delta_f$	$\delta_f = \frac{3\tau_f}{2G_a} + \frac{3}{4} \delta_1$
Fernando [41]	$\tau_f = 0.9f_{t,a}$	0.081 (mm)	0.80 (mm)	
He and Xian [44]	$\tau_f = 0.5f_{t,a}$	0.08 (mm)	$\delta_2 = \frac{G_f}{\tau_f}$ With $G_f = 10.65t_a^{1.745} R^{0.437}$	$\delta_f = \delta_1 + \delta_2$

2.8.2 Models based on adhesive failure

Very limited works have been conducted on realistic representative specimens for reinforced configuration, such as double-reinforced specimens. As stated before, this specimen fails under adhesive failure even with adequate surface treatment. In this section, the very limited works were conducted to estimate the adhesive joint strength based on experimental tests that failed under adhesive mode.

One attempt was made by Bicciarelli et al. [45] to predict the debonding of double shear lap specimens that failed under adhesive mode. The authors used the analytical formulation based on both the stress approach and fracture mechanics approach derived by ref. [46] for concrete elements. The main idea was to equate the closed-form solution based on stress analysis (see equation (9)), with fracture mechanics one (equation (10)) to derive a formulation to estimate strain energy release rate Gf , shown equation (11). However, the assumption considered for this method was very similar to the ones used for the bond-slip approach by ignoring completely peeling forces and considering only fracture mode II.

$$P_{max} = \alpha \frac{-2b_a \tau_{2,max}(\delta + 1)}{\lambda \cdot \delta} \quad (9)$$

$$P_{max} = \alpha \frac{2}{\delta} A_s \sqrt{\frac{E_s}{t_s} \cdot G_{2,max} \cdot (\delta + 1)} \quad (10)$$

$$G_f = \frac{t_a}{2G_a} \tau_{a2,max}^2 \quad (11)$$

Where t_a is the thickness of the adhesive and G_a is the shear modulus and $\tau_{a2,max}$ is assumed material dependent.

Haghani and Al-Emrani [36] tested the applicability of the stress-based and the fracture mechanics analytical solution presented in the paper of Bocciarelli et al. [45], with experimental results. The stress-based analytical solution was calibrated using double reinforced small-scale specimens (not from bulk adhesive material as proposed by Bocciarelli et al. [45]). Then it was verified against experimental data on beams. The results indicated a fairly good agreement between the experimental and analytical methods. However, in this case, the specimens used for calibration of the model were very similar to the ones applied to beams, more comparison should include the different geometrical singularities that could exist in real applications. For the fracture mechanics analytical formulation, the authors compared the upper bound load obtained using this method and the strength of the adhesive obtained from fracture mechanics tests for mode I as recommended by CIRIA guidelines since it represents the minimum energy to cause failure. As a result, it was shown that the estimated fracture energy release rate obtained from this analytical solution was lower than the experimental results. This was attributed to the mode failure where the fracture in the interface has a lower critical energy release rate than the mode I of the adhesive material.

Mapping the real distribution of the stress state inside the adhesive joint is a very difficult task. Therefore, Haghani and Al-Emrani [21] abandoned the principle of relating the strength of the adhesive joint to the stress or strain demands in critical regions. They proposed a model that considers the axial force in the laminate at a certain distance from the stress concentration zone as the global parameter which attains a critical value when failure of the joint takes place. This value can be estimated only with experimental results taken from representative specimens that reflect the stress state of the real structural elements. For beams, after a comprehensive numerical study that compared the stress state inside the adhesive joint of a beam and small scale specimens namely: single shear lap, steel bar reinforced with CFRP in both sides, and

double strap joint. It was found that the reinforced bar specimen reflects very well the behavior of the adhesive joint for the reinforced beam. Except for a slight difference that was caused by the bending effect in the beam. Thus a corrector factor α was used to consider this effect. The failure criterion is formulated as:

$$F_{LB} \leq \alpha F_{LP} \quad (12)$$

Where F_{LP} is the force in the laminate of the representative specimen, which can be estimated using beam theory. More details regarding α can be found in their paper [21].

$$F_{LP} = P_u \frac{(EA)_L}{2(EA)_L + (EA)_s} \quad (13)$$

A comparison between measured and predicted strength of beam specimens showed very good agreement, as illustrated in Figure 2-12.

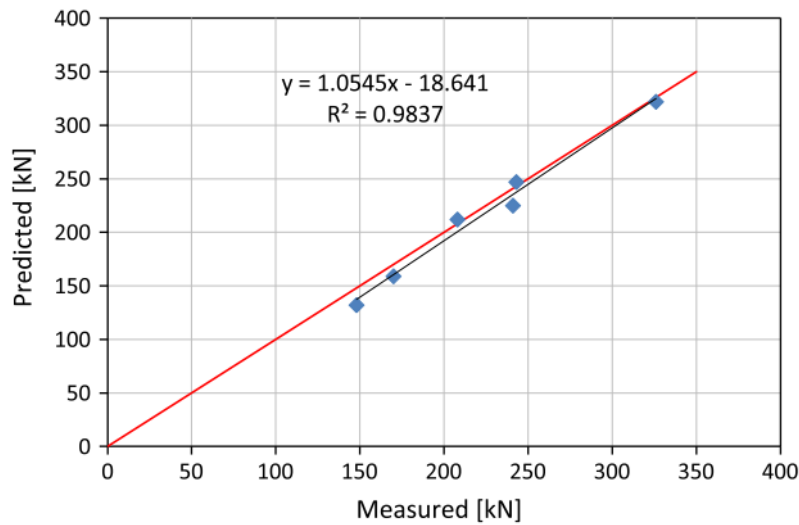


Figure 2-12: Measured and predicted strength

2.9 Influence of fatigue loading on the bonding behavior

Miller [11] strengthened the full-scale bridge girder with CFRP. To check if the fatigue will be limiting the design, he tested seven double reinforced specimens under fatigue loading with a stress range 82.7MPa. The results showed that the specimens could sustain 2.55 million cycles without failure, which fulfills the

requirements of AASHTO category C detail. The tests were stopped after reaching this fatigue threshold since the goal of the study was achieved.

Matta et al. [22] tested double-sided reinforced specimens and double strap joint specimens under monotonic loading after being fatigued for 1 million cycles. The fatigue stress range was equal to 83 MPa with $R=0.05$ (minimum and maximum load 0.85KN and 17KN) and at a frequency of 10 Hz. The epoxy adhesive, Sikadur 30 was used. The results showed that fatigue loading had little influence on the overall response. For the selected fatigue loading range, the specimens sustained a larger number of cycles than the specified by Eurocode 3 part 9 [47] categories 36 and 50 for cover plates welded to tension members.

Deng and Lee [9] tested small-scale steel beams bonded with CFRP. The backface strain technique was used to detect crack initiation and monitor the crack. The epoxy resin SIKADUR 31 was used. The authors noticed that the crack initiated in mode 1. It was found that the specimens subjected to 40KN and less did not fail even after 4 million cycles. Therefore, the fatigue threshold was considered as 40KN corresponding to 23.8MPa for the adhesive which to 30% of the ultimate static loading. The fatigued specimen after 4 million cycles was subjected to static loading until failure. The strength was marginally lower with 133 instead of 135MPa. It was observed that fatigue behavior is much more influenced by the level of maximum load, compared to the stress range.

Bocciarelli et al. [24] tested double reinforced specimens under fatigue loading to investigate the fatigue performance of the adhesive joints. CFRP plates Carbodur M614 with $e=1.4\text{mm}$ and $E=200\text{GPa}$ were bonded using Sikadure 30. The surface preparation was prepared using an abrasive disk then degreased with xylene-based solvent. Then, in order to increase the surface roughness very fine sandpaper (grit P240) was used. However, As stated before this is not the best method to ensure good bonding between the steel substrate and the adhesive. The results showed that the dominant mode of failure observed in the specimens is adhesive failure at the steel-adhesive interface. An asymmetric behavior between both CFRP ends and between both sides of the specimens was generally observed. Debonding initiated in different manners at plate ends and induced bending stress. The authors proposed fatigue S-N curves for the adhesive joint.

Three failure states were assumed that corresponds to 2%, 5%, and 15% of reduction in specimens' stiffness. With a visual inspection, it was assumed that a 2% reduction in stiffness corresponds to fatigue crack initiation in the adhesive joint. The stress ratio of the loading was equal to $R=0.4$. Tests were conducted under the maximum stress =140, 167, 200, 267MPa and stress range =83, 100, 120, 160 MPa, respectively. The reduction in stiffness versus the number of cycles is shown in Figure 2-13, where it can be seen that the specimen FT4 subjected to the highest stress ratio has the lowest fatigue performance. The level of maximum load affects significantly the fatigue performance of the adhesive joint.

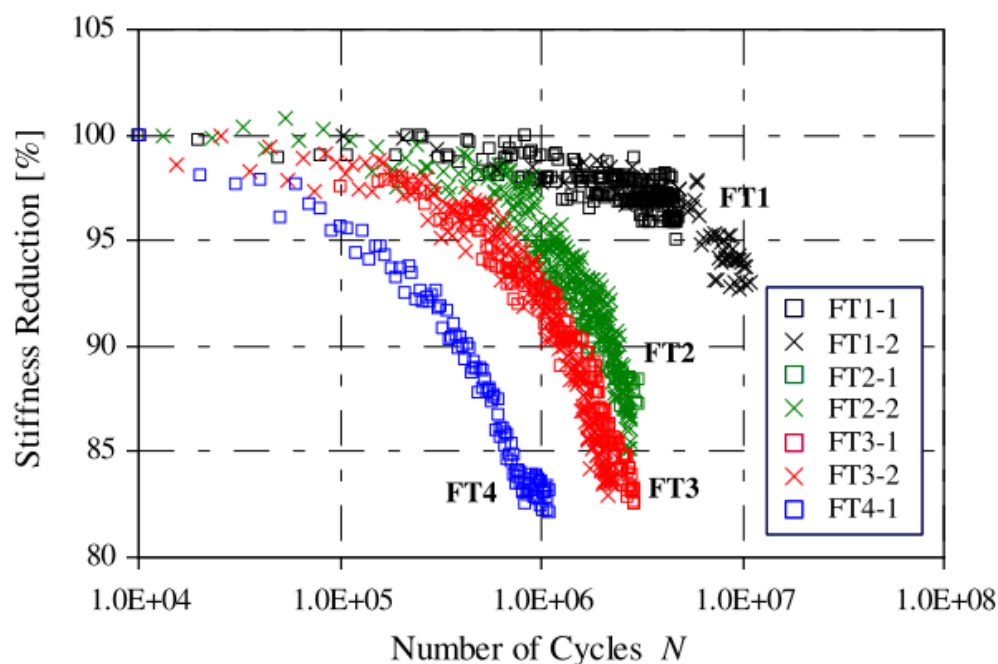


Figure 2-13: Fatigue stiffness reduction of CFRP reinforced specimens[24].

Finally, Bocciarelli et al [24] derived SN-curves according to the level of stiffness reduction. Figure 2-14 shows that the fatigue performance of the adhesive joint fits well the S-N curves, the formulations for each level of damage are plotted within the figure. By making an analogy with fatigue strength curves of Eurocode 3 part 1-9 [47]. This method of retrofitting is competitive with welded cover plates, category 90, as shown in Figure 2-14.

Colombi and Fava [23] followed the same approach as Bocciarelli et al. [24], but they tested double strap joint specimens instead. The effect of the stress ratio R was studied by applying fatigue loading ratios $R=0.1$ and $R=0.4$. The epoxy resin SIKADURE 30 was used. The tests were completed until the complete failure. The dominant mode of failure was mixed between CFRP/adhesive interface debonding and CFRP delamination. The main conclusions drawn were that the stress ratio has a marginal effect on the fatigue limit. The fatigue limit can be conservatively estimated equal to a stress range of $\Delta\sigma = 75\text{MPa}$ was conservatively proposed. It corresponds to the stress range in the steel plate which produces 95% of reduction in stiffness after 6 million cycles. The results plotted in Figure 2-15 showed that the fatigue resistance of Double strap joints is comparable to the one of welded joints (Eurocode 3 design category 40–80). The authors recommended to conduct additional testing in order to provide an additional dataset of fatigue data in order to provide a more accurate estimation. Of the fatigue life.

Doroudi et al. [31] tested single shear pull-off specimens under monotonic and cyclic quasi-static loading. The epoxy resin SIKADURE 30 was used. All the specimens failed under cohesive mode. The results showed that the cyclic loading has a negligible influence on the bond strength. It was found that the stiffness of the load-displacement tends to decrease with the number of cycles, indicating an increase of damage that is highly dependent on the amplitude of the applied loading.

Wu et al. [48] tested double strap joints between UHM CFRP and steel to check the bond strength under static loading after being fatigued by several cycles. A nonlinear adhesive was used with a tensile strength of 28.6MPa, tensile modulus 1.9 GPa, and elongation at break 2.4%. The tests were conducted under stress ratio $R=0.1$, with a maximum load equal to 20%, 30%, 40% 50%, and 60% of the bond strength measured in control specimens. Some specimens that survived the fatigue loading were compared with control specimens under static loading, and the results showed a decreasing trend for the bond strength when the maximum load increases. However, the reduction of bond strength is small with the maximum reduction being less than 4.5% compared to the control specimens. The authors observed that the fatigue had little effect on the bond strength for UH CFRP bonded specimens and the limited damage caused by fatigue was

only localized in the stress concentration zone called the damage zone. Noting that the UH CFRP is a special case because the failure mode is usually localized in the CFRP plates.

Zhang et al. [49] conducted an experimental study on interfacial bond behavior between CFRP sheets and steel plates under fatigue loading using a double strap joint. The epoxy resin used has a tensile strength equal to 41.3MPa, $E=2737.3\text{MPa}$, elongation at failure= 1.7%, and shear strength equal to 17.7MPa. The objective of this study was to investigate the effect of the level of maximum and minimum applied load on the fatigue performance of the adhesive joint. The results showed that fatigue life decreases significantly with increasing maximum applied load and increases with increasing minimum load level.

2.10 Numerical modeling adhesive joints

The FE method is a powerful tool to analyze complex geometries or conduct parametric studies that require a large number of simulations that cannot be reasonably tested in the laboratory. When it comes to the modeling of adhesive joints, it is essential to have a clear picture of how these joints behave, what parameters influence the response, and what are the consequences of making any assumptions. It also requires experience and careful interpretation of the results, otherwise, it could provide misleading results due to several shortcomings such as numerical errors and convergence problems around singular stress fields.

In general, the adhesive joint can be modeled using continuum finite element models, fracture mechanics models, and damage mechanics models.

The continuum approach is good to have a clear picture of the stress state in the adhesive bond line however, it suffers from stress singularities points which makes the estimation of failure difficult. According to Haghani [15], stress singularities in adhesive joints occur at geometrical and material discontinuities. At each adherend-adhesive interface can be regarded as material discontinuity. Possible positions of stress singularities are shown in Figure 2-16.

The severity of the singularity can be observed by the level of stress developed around the singular point. It is mesh-dependent, as illustrated in 2-17, where the effect of mesh refinement on shear stress at the steel-adhesive interface. Large stress values due to singularities can be leveled out by the use of elastic-plastic models.

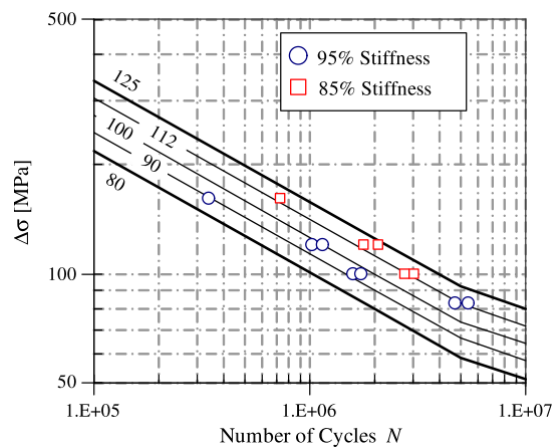


Figure 2-14: Eurocode S-N curves and adhesive joint fatigue performance [24].

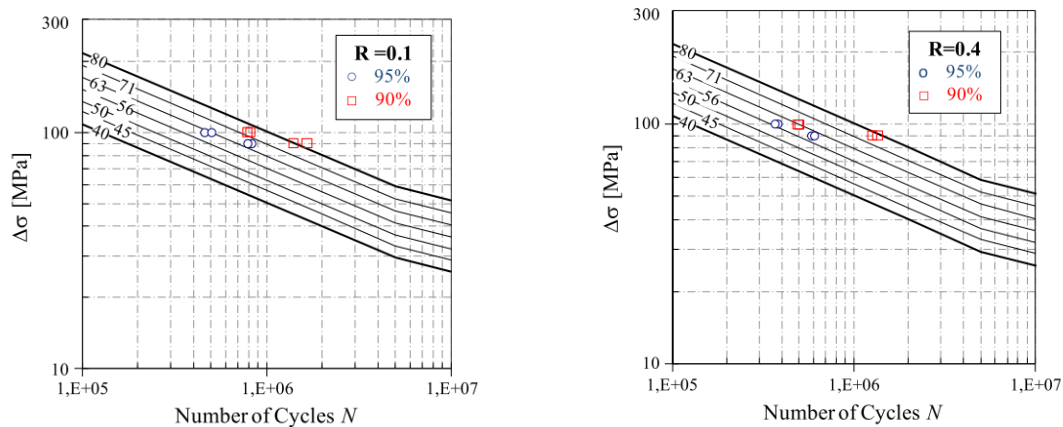


Figure 2-15: Eurocode fatigue strength curves versus double strap joint bonded CFRP with epoxy SIKADURE 30 [23].

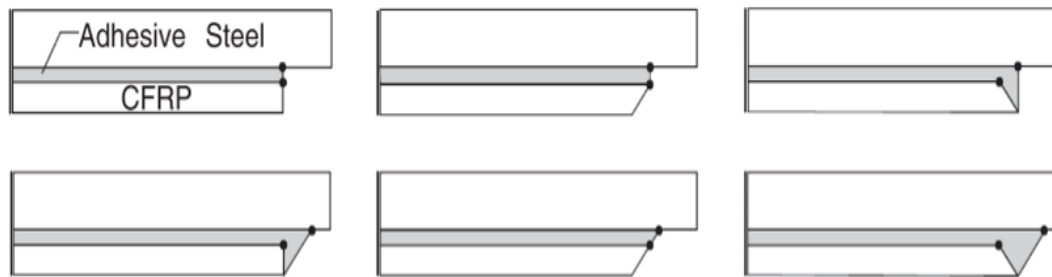


Figure 2-16: Possible positions of stress singularities [15].

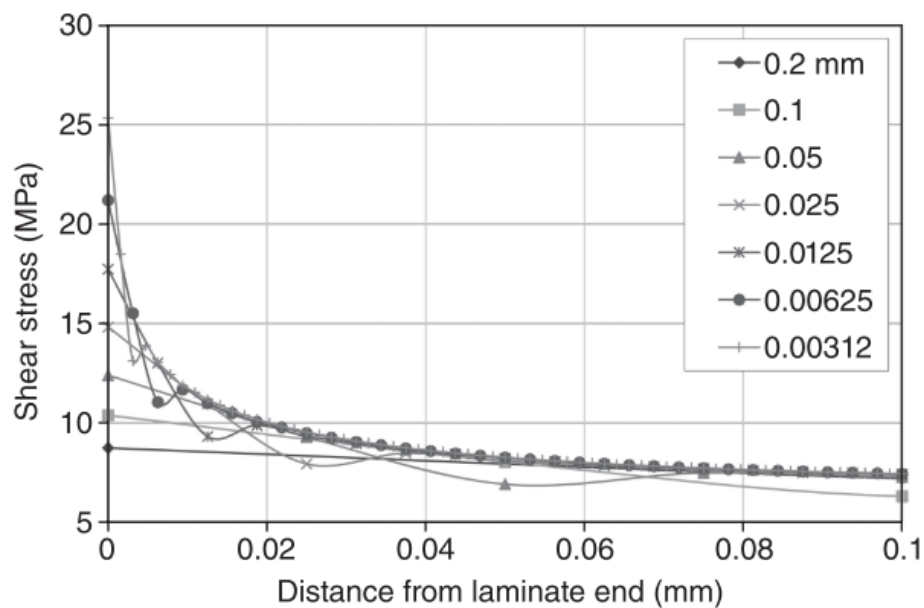


Figure 2-17: Mesh size dependence on shear stress of steel-adhesive interface [15].

Similar stress singularity issues can be found when fracture mechanics methods such as the virtual crack closure technique are used.

The adhesive layers are most commonly modeled using cohesive zone models (CZM) which are based on damage mechanics. The behavior of the CZM model is represented by the traction separation law (TSL) as shown in Figure 2-18, which takes into account the elastic behavior, the damage initiation, and propagation through the reduction of stiffness and ultimate strength until reaching the complete failure.

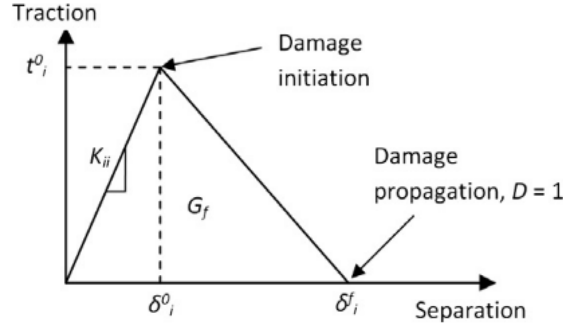


Figure 2-18: Traction separation law (TSL).

The TSL can be used for single-mode or mixed-mode conditions. In commercial software, several mixed-mode criteria can be found. The quadratic nominal stress law is the most commonly used criterion which is presented in equation (14).

$$\left\{ \frac{(t_n)}{t_n^0} \right\}^2 + \left\{ \frac{(t_s)}{t_s^0} \right\}^2 + \left\{ \frac{(t_t)}{t_t^0} \right\}^2 = 1 \quad (14)$$

Where, t_n , t_s , and t_t are stresses in the normal and parallel to the adhesive layer directions, t_n^0 , t_s^0 , and t_t^0 are the adhesive strengths in each direction. Once the damage is initiated, the damage propagation follows, where the full failure of each element is controlled by the fracture energy release rate G . Again several criteria can be found to account for mixed-mode conditions. However, the Benzeggagh–Kenane (BK)[50] criterion is commonly suggested to consider the mix mode effect and its critical fracture energy G_{mc} .

$$G_{IC} + (G_{IIC} - G_{IC}) \left(\frac{\beta^2}{1 + 2\beta^2 - \beta} \right)^\eta = G_{mc} \quad (15)$$

Where:

$$\beta = \frac{\delta_{shear}}{peel + \delta_{shear}} \quad (16)$$

G_{IC} and G_{IIC} are the mode I and Mode II fracture toughness. The parameter η is chosen to be equal to 1.5, see ref. [26]. The parameter β describes the mixed mode state based on current values of peel and shear stress. The softening part of the TLS curve is defined by a linear damage parameter D .

$$D = \frac{\delta_m^f(\delta_m^{max} - \delta_m^0)}{\delta_m^{max}(\delta_m^f - \delta_m^0)} \quad (17)$$

With

$$\delta_m^f = \frac{2G_{mc}}{\tau_m^0} \quad (18)$$

where τ_m^0 is the mixed-mode effective traction at damage initiation.

2.11 Summary and discussion

This chapter summarized issues related to the basic fundamental behavior of the CFRP/steel adhesive joints, experimental methods used in research, available analytical bond strength models, and numerical modeling techniques commonly used for design purposes.

It was shown throughout this chapter that fracture mechanics tests are rarely adopted in civil engineering applications and mostly strength failure tests are used, which consists of single shear lap joint and double strap joints. These types of specimens can be used for comparison purposes and give a good indication of the performance of the adhesive joint. They can be used as generic specimens representative of a cracked or notched structural component repaired with CFRP having stress concentration in the notched region. However, they do not represent the behavior of tensile members or steel beams that do not feature any discontinuity. The rationale behind this is that the stress transfer in cracked/notched specimens starts from the middle of the CFRP patch whereas in non-cracked/notched situations the stress transfer starts from the end of the overlap. This changes completely the behavior of the adhesive joint.

Brittle adhesives are mainly used in construction-related research. For steel structures, a more ductile and tough adhesive should be used.

The analytical and empirical models used in the literature to evaluate the bond strength have been reviewed. It was shown that the assumption of only mode II in the empirical bond-slip model is not a realistic assumption for steel structures.

The influence of the fatigue loading on the adhesive bond strength is mainly dependent on the maximum loading. Fatigue maximum loading lower than 30% of the ultimate static load is shown to have a negligible effect on the strength of the joint.

Numerical techniques to model adhesive joints are also reviewed. It was shown the continuum models can give a clear picture of the stress state in the adhesive layer. However, they strongly suffer from stress singularities. Therefore, cohesive zone models based on damage mechanics theory that overcome this issue are more appropriate to simulate the behavior of adhesive joints.

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Chapter 3: Experimental and numerical investigation on the behavior of CFRP/Steel adhesive joints

3 Experimental and numerical investigation on the behavior of CFRP/Steel adhesive joints

Abstract

This chapter presents an experimental and numerical study on the behavior of CFRP/Steel adhesively bonded double strap joints (DSJ). The selection of the adhesives was based on the tensile bulk tests, where two rigid adhesives and two ductile adhesives were selected. A total of 50 DSJ specimens were tested under tensile loading. The digital image correlation (DIC) was used to measure the deformation on the back face of the CFRP. The influence of several variants on the strength of the joints was investigated which includes: adhesive type, adhesive thickness, CFRP elastic modulus, CFRP length, surface treatment, and steel thickness. A numerical model validated with experimental results is developed. The results revealed that ductile adhesives are more suitable for CFRP/steel joints and the steel thickness has a significant influence on the behavior of the adhesive joint.

3.1 Introduction

The repair of aging metallic infrastructures is a worldwide issue [1], [2]. A large number of bridges in Europe, the USA, and Japan has more than 100 years old and faces problems of aging such as fatigue and corrosion that require particular attention [3]–[5]. The replacement of these structures is economically unfeasible. Therefore, the development of effective repair solutions is of paramount interest.

The application of bonded carbon fiber reinforced polymers (CFRP) patch to repair deficient metallic structural elements is a judicious repair solution for aging metallic infrastructures. However, this technology is still a niche field that deserves a lot of research attention. This study presents an experimental and numerical study on the behavior of adhesively bonded Steel/CFRP joints. Several variants are investigated including the influence of the adhesive type, CFRP stiffness, surface preparation, CFRP length, steel thickness.

3.2 Background

The application of bonded CFRP is a strengthening technique with great potential. The CFRP can reduce the stresses in the metallic structural elements without altering their original mechanical and fatigue properties as in the case of traditional repair methods that involve drilling or welding. This repair technology is well established in the aeronautical industry for more than three decades [6], where a bonded composite patch is commonly applied on the defected/cracked area.

In the context of the repair of metallic infrastructures, several research studies focused on the characterization of the behavior of CFRP/Steel adhesive joints [7]–[11] investigating several factors that can influence the performance, among them are the surface preparation of the adherends [7], [11],[12], the stiffness of the CFRP [14], mechanical properties of the adhesive [7], [15], [16], the thickness of the adhesive layer [7], [17], and the bond length [18]–[20].

However, most of the previous research studies used brittle adhesives which were developed for concrete structures. Although these brittle adhesives can bond well to steel substrates, they may not provide the best performance. Yu et al [7] compared Steel/CFRP joints with different adhesives. Joints with Araldite 420A/B (relatively tough adhesive) achieved significantly higher strength than the brittle ones, however, they failed under delamination failure mode, which is usually undesired by researchers in the literature. Hence, the authors [7] did not consider joints with Araldite 420A/B for the derivation of the design rules. Joints with ductile tough adhesive achieve high strength, however, the traditionally desired cohesive failure becomes almost unachievable due to the high fracture toughness of the adhesive. The ubiquitous empirical bond-slip models adopted in civil engineering-related research are only limited to cohesive failure [8]. More studies focusing on other failure modes such as the CFRP delamination should be addressed in the future [21]. The mechanical properties of the adhesive have a significant influence on the performance of the steel/CFRP adhesive joints. Recent tough adhesives with high strength and ductility are more appropriate for bonding metallic substrates [22]. Very recently, Sika [23] developed a moderately tough adhesive (i.e. Sikadure 370) for the repair of steel bridges affirming that the highly brittle 2C-epoxy adhesives used in construction practice are more suitable for concrete and other cementitious systems but for more ductile substrates such as steel toughened ductile

adhesives should be used. Kasper et al. [2] characterized the behavior of the Sikadure 370 and compared it with the most common brittle adhesive Sikadure 30. The results revealed that the tough adhesive is more suitable for the strengthening of steel bridges due to its high strength and durability.

Besides, previous research studies [7]–[9], [13], [15], [16], [18]–[20], [24] that focused on the characterization of CFRP/steel bonded joint designed their specimens in order to avoid yielding in the steel part, which is reasonable if the objective is to characterize the behavior of the adhesive in a bonded joint. However, yielding may occur in practice. So, it is also important to use more representative steel sections and examine their behavior.

3.3 Objectives

This study presents an experimental and numerical study on the behavior of adhesively bonded Steel/CFRP joints. Firstly, experimental coupon tests on several adhesive types are conducted to compare their basic mechanical properties. Then, adhesively bonded CFRP/steel double strap joints, representative of a composite patch repair over the cracked region [25], [26], are tested under static tensile loading. Then several variants are investigated including the influence of the adhesive type, CFRP stiffness, surface preparation, CFRP length, steel thickness. The digital image correlation (DIC) is used to measure the deformation on the back face of the CFRP. A continuum finite element model calibrated with experimental data is developed in ABAQUS. Based on the experimental and numerical results a comprehensive discussion on the behavior and the design of steel/CFRP joints is provided.

3.4 Experimental program

3.4.1 Materials

3.4.1.1 Adhesives

Dog-bone specimens were manufactured and tested under tensile loading to compare the behavior of the different adhesives. The specimens were designed according to ASTM D638-14 [27] category 5 which is recommended for the comparison between different adhesive types. A total of 6 six different adhesives were selected for testing. They consist of:

- **Sikadur 30. (SK-30):** an epoxy-based linear brittle adhesive commonly used in the construction industry and related research studies.
- **S&P HP220:** an epoxy-based linear rigid adhesive commonly used in the construction industry.
- **Sika Power 1277 (SK):** an epoxy-based ductile adhesive developed for the automotive industry but can be found in the research literature for steel structures.
- **Araldite 2052-1. (AR):** an acrylic-based ductile adhesive developed for harsh environments. It has high resistance to humidity and high temperature. The manufacturers claim that it requires no or limited surface preparation.
- **Araldite AW4858/HW4858. (AW):** An epoxy-based ductile adhesive developed for different mechanical engineering industries. It has high peeling resistance and shear resistance.
- **Araldite LY5052/Aradur 5052CH. (LY):** an epoxy-based linear adhesive developed for CFRP impregnation and wet-layup technique.

The specimens were manufactured using an acrylic mold placed in between two flat rectangular acrylic plates and compressed with weights to ensure constant thickness, push out air bubbles, fill the gaps with adhesive, and evacuate extra volume of the adhesive. The specimens were left to cure for at least 7 days at room temperature. The adhesive specimens were tested in MTS 831-02 Elastomer test system, as shown in Figure 3-1. The strain measurements were taken using clip gauges. The specimens were subjected to static tensile loading until complete failure. The speed of the loading was 1mm/minute except for very brittle adhesives that failed before one minute, it was reduced to 0.5mm/minute. Since the ASTM D638 recommends using a speed between 0.5-5mm/minute given that the duration of the test shall be more than 1 minute.

The basic average material properties of the adhesives are presented in Table 1, including Young's modulus (E), the maximum stress, maximum deformation before failure, and toughness which is measured as the area under the stress-strain curve. Figure 3-2 illustrates the stress-strain curves of the different adhesives. The adhesive SK30 reached high strength (i.e. 35MPa) and stiffness but very low strain capacity up to 0.3%. The adhesive HP220 has a lower stiffness than SK30 and reaches roughly similar strength but with doubled deformation capacity. The adhesive LY has an intermediate rigidity

and very high strength with an average maximum of 57.6MPa . The ductile adhesives AW, AR, and SK exhibit a lower stiffness than the rigid adhesives. The adhesive AW reached a high strength of 35MPa and significantly longer deformation at failure with an average factor of 64 and 48 compared to rigid adhesives SK30 and HP220, respectively. The adhesive SK reached an average strength of 30.8MPa but did not exhibit large plastic deformation. The adhesive AR reached the lowest strength with the value of 23.3MPa but the largest deformation capacity of 18.4%. Note one specimen that failed prematurely, as shown in Figure 3-2, was excluded from the calculation of the mechanical properties. The premature failure was mainly due to a large void that was found inside the material in the failed section.

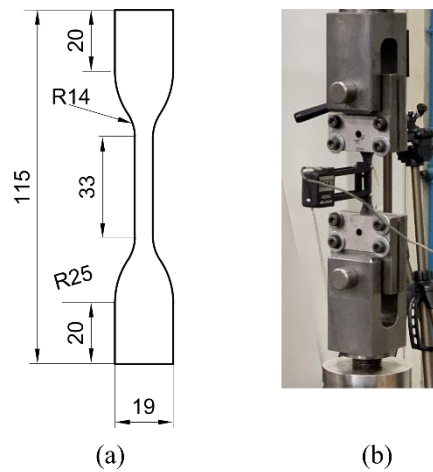


Figure 3-1: Adhesive dog-bone tensile test in MTS 831-02.

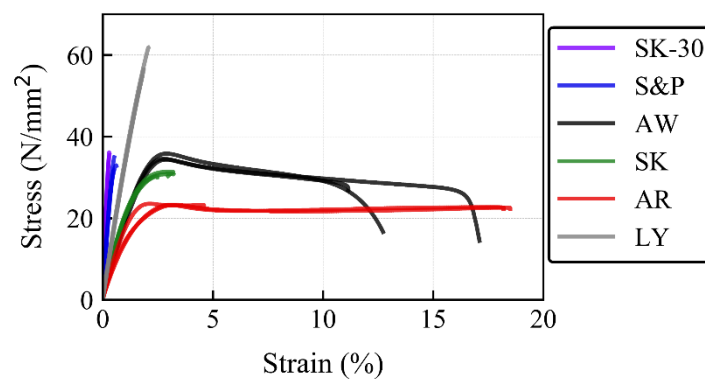


Figure 3-2: Stress strain curves of adhesive dog-bone tensile tests.

Table 3-1: Adhesives average mechanical properties.

Adhesive	E (MPa)	Max stress (N/mm ²)	Max Deformation (%)	Toughness (N/mm ²)
SK-30	12915	35.03	0.28	0.058
HP220	7600	33.16	0.54	0.108
AW	1350	34.9	13.6	3.9
AR	1240	23.3	18.4	3.88
LY	3255	57.6	1.9	0.59
SK	1695	30.8	2.97	0.64

3.4.1.2 CFRP

Two types of CFRP were used in this study: (a) prefabricated pultruded unidirectional laminates (i.e. S&P reinforcement HM (200/2000)); and (b) high modulus CFRP sheets (S&P C-Sheet 640) that were impregnated with the adhesive by the authors in the lab using the wet layup technique. The Araldite LY5052/Aradur 5052CH. (LY) was used for the impregnation of the CFRP sheets. Two methodologies were applied in the wet layup technique which consists of:

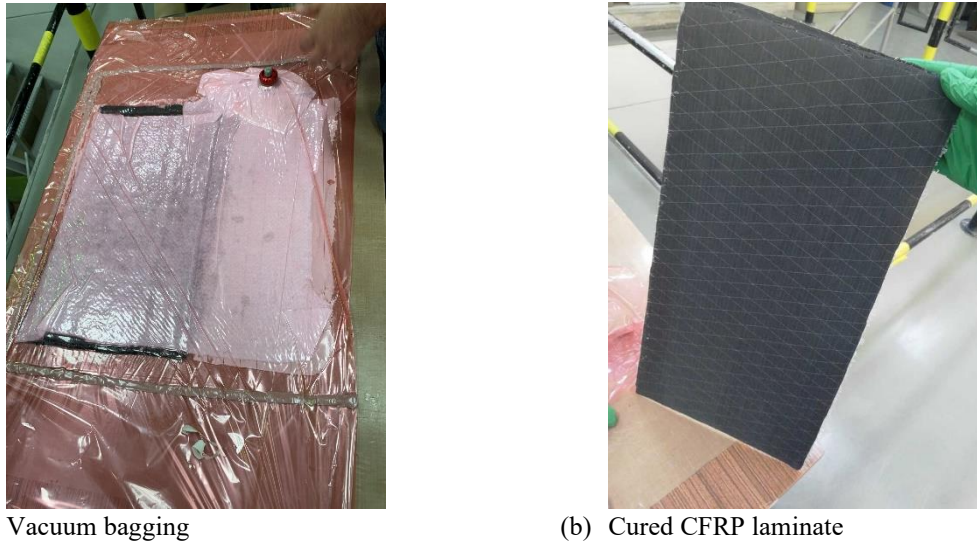
Method 1: Impregnating the CFRP sheets with the adhesive LY5052/Aradur 5052CH during the manufacture of the CFRP/Steel double strap joint. In this case, the adhesive is used to wet the CFRP sheets and the steel bars with CFRP simultaneously. This technique is commonly used in the literature.

Method 2: Prefabricating the CFRP laminates using the vacuum bagging technique as shown Figure 3-3. The CFRP laminates were cut in the required dimensions. In this case, any adhesive can be used to bond the CFRP/steel adhesive joint as the CFRP is prefabricated.

The nominal mechanical properties of the CFRP provided by the manufacturer are presented in Table 3-2.

Table 3-2: Basic mechanical and geometrical properties of the CFRP provided by the manufacturer.

Type	Young modulus (GPa)	Ultimate stress (MPa)	Elongation at break (%)	Thickness (mm)
S&P HM 200/2000	205	2800	1.3%	1.4
S&P C-Sheet 640	640	2600	0.4%	0.168



(a) Vacuum bagging

(b) Cured CFRP laminate

Figure 3-3: Manufacture of high modulus CFRP laminates.

3.4.1.3 Steel

Steel bars made with two thicknesses (i.e. 6mm and 10mm) made of European hot rolled carbon steel S275 were used in this study. The measured mechanical properties are shown in Table 3-3.

Table 3-3: Measured mechanical properties of S275.

Material	Yield stress (MPa)	Ultimate stress (MPa)	Elastic modulus (GPa)
S275 (10mm)	350	439	200
S275 (6mm)	324	458	200

3.4.2 Fabrication of double strap joints specimens

Adhesively bonded CFRP/Steel double strap joint specimens (DSJ) are used to assess the behavior of the adhesive interface between steel and CFRP. DSJ specimens are commonly adopted since they are representative of a composite patch repair over a cracked region [25], [26]. A typical double strap joint, as shown in Figure 3-4, is constituted by two separate steel bars joint together with a bonded CFRP patch on both sides.

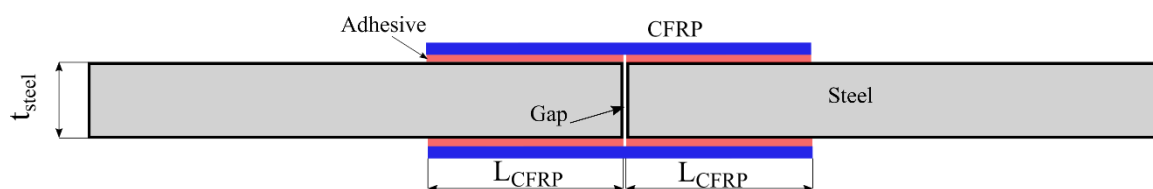


Figure 3-4: Schematic view of double strap joint specimens.

The manufacture of the specimens involved mainly two steps:

- (a) Surface preparation of the steel and CFRP.
- (b) Application of the adhesive bonding between CFRP and steel in the mold.

3.4.2.1 Surface preparation

The surface preparation is a key step for the successful bonding of structural adhesives [28]. Except for two specimens that had the surface treated only with sand paper using an angle grinder, all the specimens had the steel substrates sand blasted in the NORBLAST machine, as shown in Figure 3-5. The alumina dioxide sand F60 with grain size between 212 – 300 *microns* was used to prepare the surface of the joint, as recommended in [12] to avoid adhesion failure in the steel-adhesive interface which usually occurs due to inadequate surface preparation. The two specimens polished using an angle grinder with sand-paper grit 80 were used to study the effect of surface treatment.

Once the specimens were polished, the surfaces were cleaned with compressed air and acetone to remove any particles left from the surface preparation operation. The CFRP surface was polished with sandpaper with grit 180 which is commonly used for composite materials. The polishing of the CFRP plates was made carefully and only in the direction of the fibers.



Figure 3-5: Sand blasting of the steel bars: (a) sand blasted steel bar; (b) sandblast machine.

3.4.2.2 Application of the adhesive bonding

The double strap joints were manufactured using aluminum molds that have been fabricated at the DEMEC workshop in the Faculty of Engineering University of Porto specifically for this application. The molds are made of two flat aluminum plates of 25 mm thickness, 1300 mm long, and 250 mm

width. The molds can accommodate 4 specimens in the longitudinal direction separated by pins, as shown in Figure 3-6. The main role of the pins is to ensure that the CFRP and the steel bars are well aligned. The preparation of the mold includes cleaning and applying the unmoulding agent to be able to unmould the specimens after the cure period. The adhesives were applied on both CFRP, and the steel then bonded together. Once, the CFRP and the steel bars were joined together, aluminum spacers were used to maintain the predefined adhesive thickness constant after closing the mold where the upper plate will apply uniform pressure on the specimens. Then, the specimens were left to cure for at least 7 days at room temperature. Once, the cure time passed, the specimens were unmoulded and the extra adhesives that leaked to the sides was polished to make sure to have the adhesive aligned with CFRP and the steel bars.

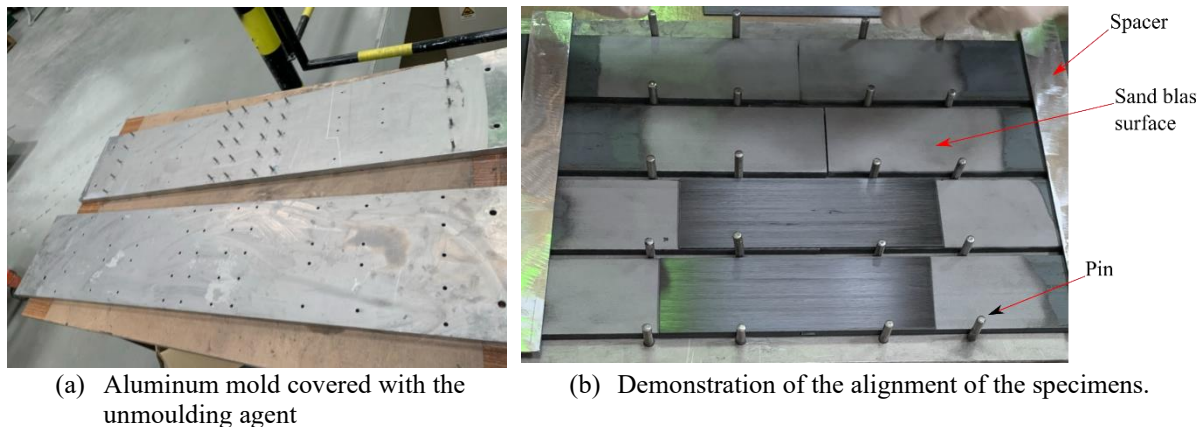


Figure 3-6: The aluminum Mould used for manufacturing the double strap joints.

3.4.3 Experimental set-up

The double strap joint specimens were tested in an ESH testing machine in the materials lab at the Faculty of Engineering University of Porto (FEUP), as shown in Figure 3-7. The specimens were subjected to static tensile loading until complete failure. The loading speed was 1 mm/minute. The strain fields on the back face of the CFRP plates and on the lateral face of the joint were captured using 2D digital image correlation (DIC). A random speckle pattern with a black background and white spots was applied on the back face of the CFRP. Note, a maximum of 100mm lens imaging was available so the lenses were controlled to achieve a sharp focus only on half of the joint.



Figure 3-7: Experimental setup.

3.4.4 Experimental testing matrix

A total of 50 specimens has been tested, the experimental matrix is presented in Table 3-4. The nomenclature of each specimen is given as follows “DSJ-CFRP type-CFRP length-Steel thickness-Adhesive name-Adhesive thickness- repetition No”. Two specimens include (AG) in the end which refers to surface preparation with angle grinding. The CFRP S&P HM 200/2000 is represented as (S&P). The acronyms H10 and H5 stand for CFRP laminates impregnated using vacuum bagging technique with 10 layers and 5 layers of high modulus CFRP sheets S&P C-640, respectively. Whereas, WH5 and WH1 stand for 5 layers and 1 layer of high modulus S&P C-640CFRP sheets applied directly on the specimens using the wet layup technique.

Table 3-4: Experimental testing matrix of steel/CFRP double strap joints.

No.	Specimen's Name	Control	Type of Adhesive	Type of CFRP	L_{CFRP} (mm)	$t_{adhesive}$ (mm)	t_{steel} (mm)	Strength (kN)	Failure mode
1	DSJ-SP-75-10-HP220-1-1	Adhesive type	HP220	S&P	75	1	10	103.8	D
2	DSJ-SP-75-10-HP220-1-2		HP220	S&P	75	1	10	99.9	D
3	DSJ-SP-75-10-HP220-1-3		HP220	S&P	75	1	10	110	D+C
4	DSJ-SP-75-10-HP220-1-4		HP220	S&P	75	1	10	106	D+C
5	DSJ-SP-75-10-AW-1-1		AW	S&P	75	1	10	155.5	D
6	DSJ-SP-75-10-AW-1-2		AW	S&P	75	1	10	161.5	D
7	DSJ-SP-75-10-AW-1-3		AW	S&P	75	1	10	168.6	D
8	DSJ-SP-75-10-AW-1-4		AW	S&P	75	1	10	153.0	D
9	DSJ-SP-75-10-AW-1-5		AW	S&P	75	1	10	161.0	D
10	DSJ-SP-75-10-AW-1-6		AW	S&P	75	1	10	166.9	D
11	DSJ-SP-75-10-SK30-1-1		SK30	S&P	75	1	10	80.0	C+D
12	DSJ-SP-75-10-SK30-1-2		SK30	S&P	75	1	10	80.0	C+D
13	DSJ-SP-75-10-SK30-1-3		SK30	S&P	75	1	10	76.6	C+D
14	DSJ-SP-75-10-SK30-1-4		SK30	S&P	75	1	10	70.8	C+D
15	DSJ-SP-75-10-SK30-1-5		SK30	S&P	75	1	10	66.7	C+D
16	DSJ-SP-75-10-SK30-1-6		SK30	S&P	75	1	10	82.1	C+D
17	DSJ-SP-75-10-AR-1-1		AR-52	S&P	75	1	10	156.2	D
18	DSJ-SP-75-10-AR-1-2		AR-52	S&P	75	1	10	177.5	D
19	DSJ-SP-75-10-AR-1-3		AR-52	S&P	75	1	10	175.1	D
20	DSJ-SP-75-10-AR-1-4		AR-52	S&P	75	1	10	175.3	D
21	DSJ-SP-75-10-AW-0.6-1	Adhesive thickness	AW	S&P	75	0.6	10	174.4	D
22	DSJ-SP-75-10-AW-0.6-2		AW	S&P	75	0.6	10	174.3	D
23	DSJ-SP-75-10-AW-1.6-1		AW	S&P	75	1.6	10	142.5	D
24	DSJ-SP-75-10-HP220-0.6-1		HP220	S&P	75	0.6	10	122.0	C+D
25	DSJ-SP-75-10-HP220-1.6-1		HP220	S&P	75	1.6	10	117.3	C+D
26	DSJ-SP-75-10-HP220-0.6-2		HP220	S&P	75	0.6	10	122.0	C+D
27	DSJ-SP-75-10-HP220-1.6-2		HP220	S&P	75	1.6	10	109.7	D+C
28	DSJ-SP-75-6-AW-1-1	Steel thickness	AW	S&P	75	1	6	121.5	D
29	DSJ-SP-75-6-AW-1-2		AW	S&P	75	1	6	107.9	D
30	DSJ-SP-75-6-HP220-1-1		HP220	S&P	75	1	6	101.4	A+D
31	DSJ-SP-75-6-HP220-1-2		HP220	S&P	75	1	6	105.2	A+D
32	DSJ-SP-30-10-AW-1-1	CFRP length	AW	S&P	30	1	10	72.5	D
33	DSJ-SP-50-10-AW-1-2		AW	S&P	50	1	10	104.4	D
34	DSJ-SP-30-10-HP220-1-1		HP220	S&P	30	1	10	39.1	C+D-
35	DSJ-SP-30-10-HP220-1-2		HP220	S&P	30	1	10	35.3	C+D-
36	DSJ-SP-50-10-HP220-1-1		HP220	S&P	50	1	10	76.9	D+C-
37	DSJ-SP-50-10-HP220-1-2		HP220	S&P	50	1	10	85.9	D+A

38	DSJ-H10-75-6-AW-1-1	CFRP type	AW	H10	75	1	6	107.8	R
39	DSJ-H5-75-6-AW-1-1		AW	H5	75	1	6	115.7	R
40	DSJ-H5-75-6-AW-1-1		AW	H5	75	1	6	105.4	R
41	DSJ-W5-75-6-LY-1-1		LY	WH5	75	1	6	76.5	R
42	DSJ-W1-75-6-LY-1-1		LY	WH1	75	1	6	21.3	R
43	DSJ-H10-75-10-AW-1-1		AW	H10	75	1	10	161.3	R
44	DSJ-H10-75-10-AW-1-2		AW	H10	75	1	10	173.3	R
45	DSJ-SK-75-6-AW-1-1		AW	S512	75	1	6	119.8	D
46	DSJ-SK-75-6-AW-1-2		AW	S512	75	1	6	105.4	D
47	DSJ-SK-75-6-HP220-1-1		HP220	S512	75	1	6	101.9	A+D
48	DSJ-SK-75-6-HP220-1-2		HP220	S512	75	1	6	101.0	A+D
49	DSJ-SK-75-10-HP220-1-1		HP220	S512	75	1	10	119.6	A+D
50	DSJ-SK-75-10-HP220-1-2		HP220	S512	75	1	10	118.7	D+C

A: Adhesive failure; C: Cohesive failure; D: Delamination mode of Failure; R: CFRP rupture. Note: for combined failure modes, the first letter is the dominant failure mode.

3.5 Results

3.5.1 Load displacement curves

Figure 3-8 (a) illustrates the load-displacement curves for the 6 repeated tests of specimens “DSJ-SP-75-10-AW-repetition No.” which are made by the tough adhesive AW4858/HW4858 (1mm thickness), normal modulus CFRP (S&P $L = 75mm$), and a steel thickness of 10mm. The maximum loads reached are between 150kN-170kN, which is very close to the yield strength of the steel bar, as shown in the figure where a load-displacement curve of a bare steel bar of the same material and geometry is plotted. Specimen “DSJ-SP-75-10-AW-6” shows a yielding flat plateau which indicates that the steel bar had yielded as it will be shown numerically in the subsequent sections. The maximum displacement reached before failure is 4.7mm. The limited ductility shown in the other repeated tests might be due to plastic deformations in the adhesive or steel yielding in regions of stress concentration which will be discussed further in the subsequent sections.

Figure 3-8 (b) presents the load-displacement curves of 4 repeated DSJ specimens “DSJ-SP-75-10-SP-repetition No.” bonded with HP220 rigid adhesive. The specimens reached strength values between 99.9 – 110kN, followed by an abrupt failure.

Figure 3-8 (c) shows the load-displacement curves of 6 repeated tests of “DSJ-SP-75-10-SK30-1- (repetition No.)” which are made by the rigid adhesive SIKADUR30 (1mm thickness), normal modulus CFRP (S&P), and a steel thickness of 10mm. The strength reached values between 65 – 85 *kN* followed by a moderate flat plateau in most cases. The flat plateau has been reported in other studies in the literature [7] for joints with Sikadur 30. The maximum strength is associated with crack initiation in the middle of the joint and the flat plateau is attributed to the crack propagation which depends on the length of the bonded area.

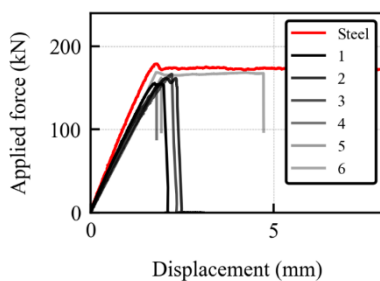
The load-displacement curves for specimens with Araldite 2052-1 “DSJ-SP-75-10-AR-1-repetition No.”, shown in Figure 3-8 (d), reached very high strength with values above 175kN except one which is considered as an outlier which reached only 156kN due large voids observed in the adhesive layer after failure. Specimen “DSJ-SP-75-10-AR-1-4” exhibit a large ductile plateau which follows the trend of the load-displacement curves of a bare steel bar as shown in the figure. The maximum displacement of 8.9mm was reached before failure which is almost double of the specimen “DSJ-SP-75-10-AW-6” made with AW4858/HW4858 adhesive that developed a ductile behavior.

Figure 3-8 (e) and (f) present the load-displacement curves for specimens made with steel bars of $t_{steel} = 6mm$ for both the rigid adhesive S&P HP220 (i.e. DSJ-SP-75-6-SP-1-repetition No.) and the ductile adhesive AW4858/HW4858 (i.e. DSJ-SP-75-6-AW-1-repetition No.), respectively. Due the lower yield strength of these steel bars ($t_{steel} = 6mm$), the DSJ specimens developed large plastic deformation before failure. Specimens made with the rigid adhesive SP HP220 “DSJ-SP-75-6-SP-1-repetition No.” reached the yielding strength of the steel bars (i.e. $\approx 100kN$) and developed ductile displacement up to 9.26mm. Whereas the specimens with the ductile AW4858/HW4858 developed much higher ductility and strain hardening. The maximum strength of the joint reached is 121kN and a maximum displacement of 33.6mm.

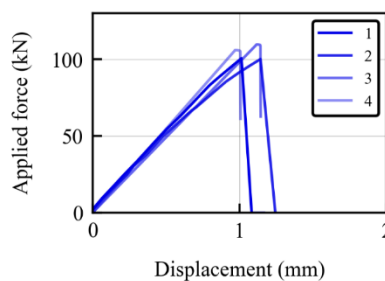
Figure 3-8 (g) illustrates the specimens with the steel of $t_{steel} = 6mm$ bonded with high modulus CFRP laminates made using the wet-layup technique. As can be seen, specimens having the CFRP laminates made with vacuum bagging and bonded with AW4858/HW4858 adhesive (i.e. DSJ-H10-75-

6-AW-1-1, DSJ-H5-75-6-AW-1-1, and DSJ-H5-75-6-AW-1-2) reached higher performance than the ones directly impregnated on the steel bars using LY5052/CH5052 epoxy resin (i.e. DSJ-W5-75-6-LY-1-1, DSJ-W1-75-6-LY-1-1). The specimens bonded AW4858/HW4858 reached the yield strength of the steel bars and developed some ductility and strain hardening. Whereas the specimens bonded with LY5052/CH5052 exhibited a linear behavior with a strength $76.5kN$ for DSJ-W5-75-6-LY-1-1 and $21.3kN$ for DSJ-W1-75-6-LY-1-1. Note the latter specimen was made with only one layer of CFRP sheets (thickness $0.168mm$) which justifies the lower strength reached. One layer of CFRP has been used previously in the literature [29], [30].

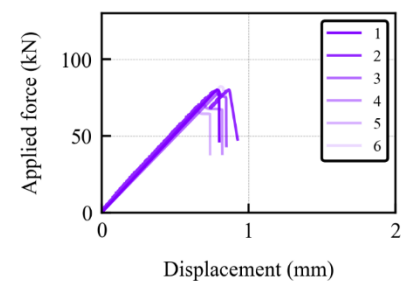
Figure 3-8 (f) shows the load-displacement curves of DSJ specimens made with the 10 layers of high modulus CFRP laminates “DSJ-H10-75-10-AW-1-repetition No.” plotted with the ones made with pultruded CFRP S&P HM(200/2000) “DSJ-SP-75-10-AW-1-repetition No.” for the sake of comparison. All the specimens are made with a steel thickness of $10mm$ and bonded with AW4858/HW4858. The specimens with high modulus CFRP reached high strength with values of $161.3kN$ and $173.3kN$, which is comparable to the ones made with pultruded CFRP.



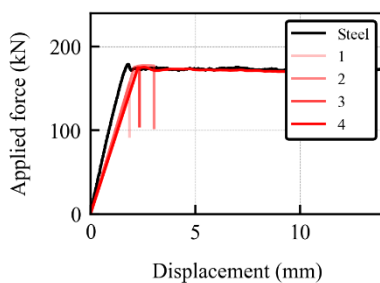
(a) DSJ-SP-75-10-AW-1-(repetition No. *)



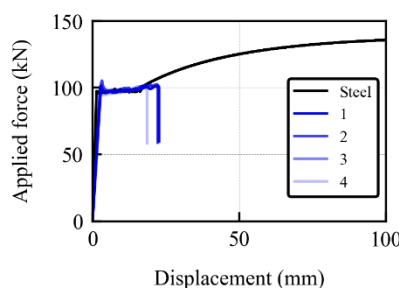
(b) DSJ-SP-75-10-SP-1- (repetition No. *)



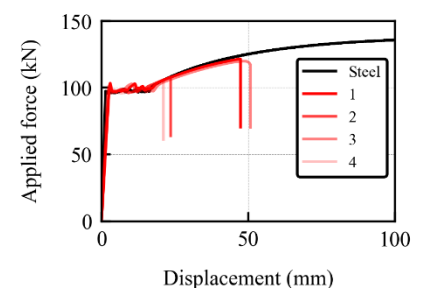
(c) DSJ-SP-75-10-SK30-1-(repetition No. *)



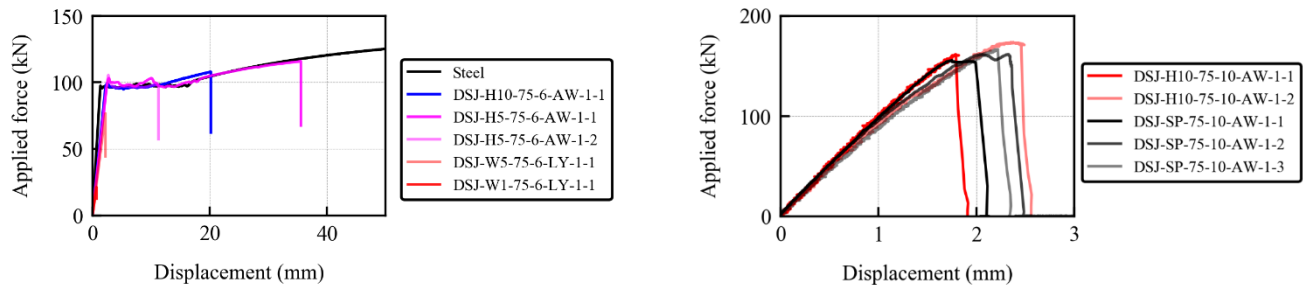
(d) DSJ-SP-75-10-AR-1-(repetition No. *)



(e) DSJ-SP-75-6-SP-1-(repetition No. *)



(f) DSJ-SP-75-6-AW-1-(repetition No. *)



(g) DSJ specimens of 6mm steel thickness bonded with high modulus CFRP made by the wet layup technique. (h) Comparison between DSJ specimens (10mm steel thickness) bonded with high modulus and normal modulus CFRP.

*Figure 3-8: Double strap joints (DSJ) load-displacement curves. *Note that repetition No. is mentioned in the legend.*

3.5.2 Failure modes

The failure modes and the inspection of the fracture surface provide important information on the behavior of the adhesive joint. According to Zhao and Zhang [31] and Majidi et al. [32] the possible mode of failures in the steel/CFRP joint are the followings: Adhesive failure at the adherends-adhesive interface caused usually by inadequate surface treatment [28], cohesive failure in the adhesive layer, CFRP rupture that occurs for high modulus CFRP due to its limited deformation capacity [31], CFRP delamination, and steel yielding.

All DSJ specimens made with pultruded CFRP and bonded with the tough adhesive AW4858/HW4858 failed under CFRP delamination failure mode, regardless of the adhesive thickness, the CFRP length, or the steel thickness (see Figure 3-9 (a), (h), and (l)). This indicates the fracture toughness of the adhesive AW4858/HW4858 is higher than the fracture toughness of the CFRP matrix where the CFRP delamination occurs.

Similarly, specimens made with pultruded CFRP and bonded with the ductile adhesive AR5052-1 failed mainly under CFRP delamination failure mode. Figure 3-9 (e) and (f) show two specimens bonded by the adhesive AR5052-1 but had different steel surface treatments. The surface treatment did not influence the failure mode of the specimens.

Specimens made with steel bars of thickness 10mm, pultruded CFRP, and bonded with S&P HP220 failed under CFRP delamination failure mode or a combination of cohesive and delamination, as shown in Figure 3-9 (c) and (d). For specimens with shorter CFRP length (i.e. 30mm and 50mm), cohesive

failure occurred more than CFRP delamination as shown in Figure 3-9 (g). For specimens with 6mm thickness, adhesive failure between the adhesive layer and the steel substrate and limited CFRP delamination governed the behavior. In these specimens, the steel bar experienced large plastic deformation as shown in the subsequent section and in Figure 3-8 (e), which create stress concentration at the interface and change the failure patch compared to the ones with a steel of 10mm thickness that did not experience plasticity.

Figure 3-8 (b) shows a typical failure mode of specimens bonded with SIKADUR30 adhesive which failed mainly under cohesive failure with limited delamination at the joint ends. This behavior has been also reported in previous studies which used SIKADUR30 [7]. This behavior indicates that SIKADUR30 has a lower fracture toughness than the CFRP delamination. The specimens with SIKADUR30 achieved the lowest strength compared to all other specimens.

Figure 3-8 (i) and (j) illustrate the failure mode of specimens made with high modulus CFRP made with the wet layup technique bonded with AW4858/HW4858 and LY5052/CH5052, respectively. Regardless the adhesive type and the number of CFRP layers (i.e. Figure 3-8 (i) 5 layers and Figure 3-8 (j) 1 layer), all specimens made with high modulus CFRP showed CFRP rupture mode of failure. The CFRP rupture failure mode occurs mainly in the low deformation capacity of high modulus CFRP (i.e. 0.4%). This observation is inline with the results of Chu et al. [14].

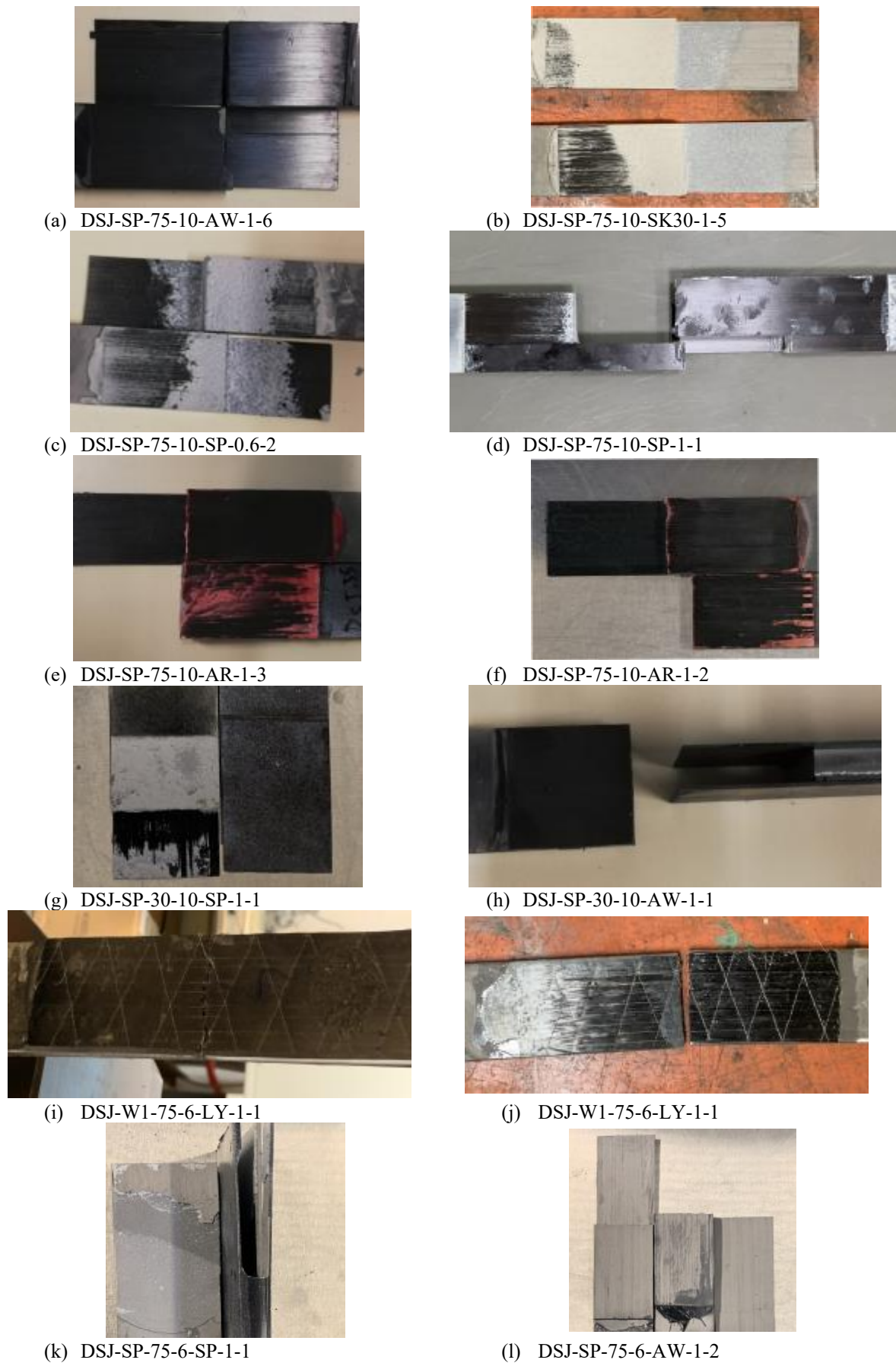


Figure 3-9: Failure modes.

3.5.3 CFRP back face strain fields using DIC

Figure 3-10 shows the strain fields contour plots on the back face of the CFRP captured using DIC for specimens made with steel thickness $t_{\text{steel}}=10\text{mm}$ and pultruded CFRP S&P HM(200/2000). Two specimens made with rigid adhesives (SIKADUR30 and S&P HP220) and two made with ductile adhesive (AW4858/HW4858 and Araldite 2052-1) are presented in Figure 3-10 (a), (b), (c), and (d), respectively. The strains are measured in the Y-Y direction parallel to the loading. Note the DIC contour plots show only half the CFRP length as captured with a DIC camera to achieve sharp focus. Figure 3-10 (a) shows the specimen made with SIKADUR 30, where the deformation concentrates near the middle of the joint until covering around 30% of the CFRP before failure, with the maximum strain that does not reach 0.4%. The specimen with S&P HP220 adhesive presented in Figure 3-10 (b), shows a larger deformed area contributing to the resistance of the joints and deformation levels exceeding 0.4% which is higher compared to specimens with Sikadur 30. At higher loading levels, the strains are not distributed uniformly which may indicate the presence of a weaker zone due to manufacturing imperfections such as voids. The specimen with the AW4858/HW4858, shown in Figure 3-10 (c), exhibits deformation at almost all the faces of the CFRP before failure which leads to higher strength as shown in Section 3.3.1.

Figure 3 11: DIC contour plots on the back face of the CFRP for specimens with steel thickness =6mm. (Note: only half of the joint in shown): (a) specimen with ductile adhesive AW4858/HW4858; (b) specimen with rigidi adhesive S&P HP220.

Figure 3-11 shows the strain fields contour plots on the back face of the CFRP captured using DIC for specimens made with steel thickness $t_{\text{steel}}=6\text{mm}$ and pultruded CFRP S&P HM (200/2000).

Figure 3 11: DIC contour plots on the back face of the CFRP for specimens with steel thickness =6mm. (Note: only half of the joint in shown): (a) specimen with ductile adhesive AW4858/HW4858; (b) specimen with rigidi adhesive S&P HP220.

(a) and (b) illustrate the contour plots for specimens made with the ductile adhesive AW4858/HW4858 and the rigid adhesive S&P HP220, respectively. Figure 3-12 shows the loading stages in load-

displacement curves corresponding to the contour plots presented in Figure 3-11. The specimen DSJ-SP-75-6-AW-1-3 made with the ductile adhesive AW4858/HW4858, shown in Figure 3-11 (a) reaches higher values of deformation than DSJ-SP-75-6-SP-1-1 made with the rigid adhesive S&P HP220 shown in Figure 3-11 (b). Unlike specimens made with steel thickness $t_{\text{steel}}=10\text{mm}$, specimens made with steel thickness $t_{\text{steel}}=6\text{mm}$ reach high deformation values at all the CFRP faces and even at the end of the overlap. Interestingly, the deformation level at the end of the overlap decrease to zero in step 3 and step 4 where the joints are subjected to large plastic deformations as shown in Figure 3-12. This indicates that the debonding starts at the end of the overlap for specimens made with steel thickness $t_{\text{steel}}=6\text{mm}$. Whereas specimens with $t_{\text{steel}}=10\text{mm}$, debonding started from the middle.

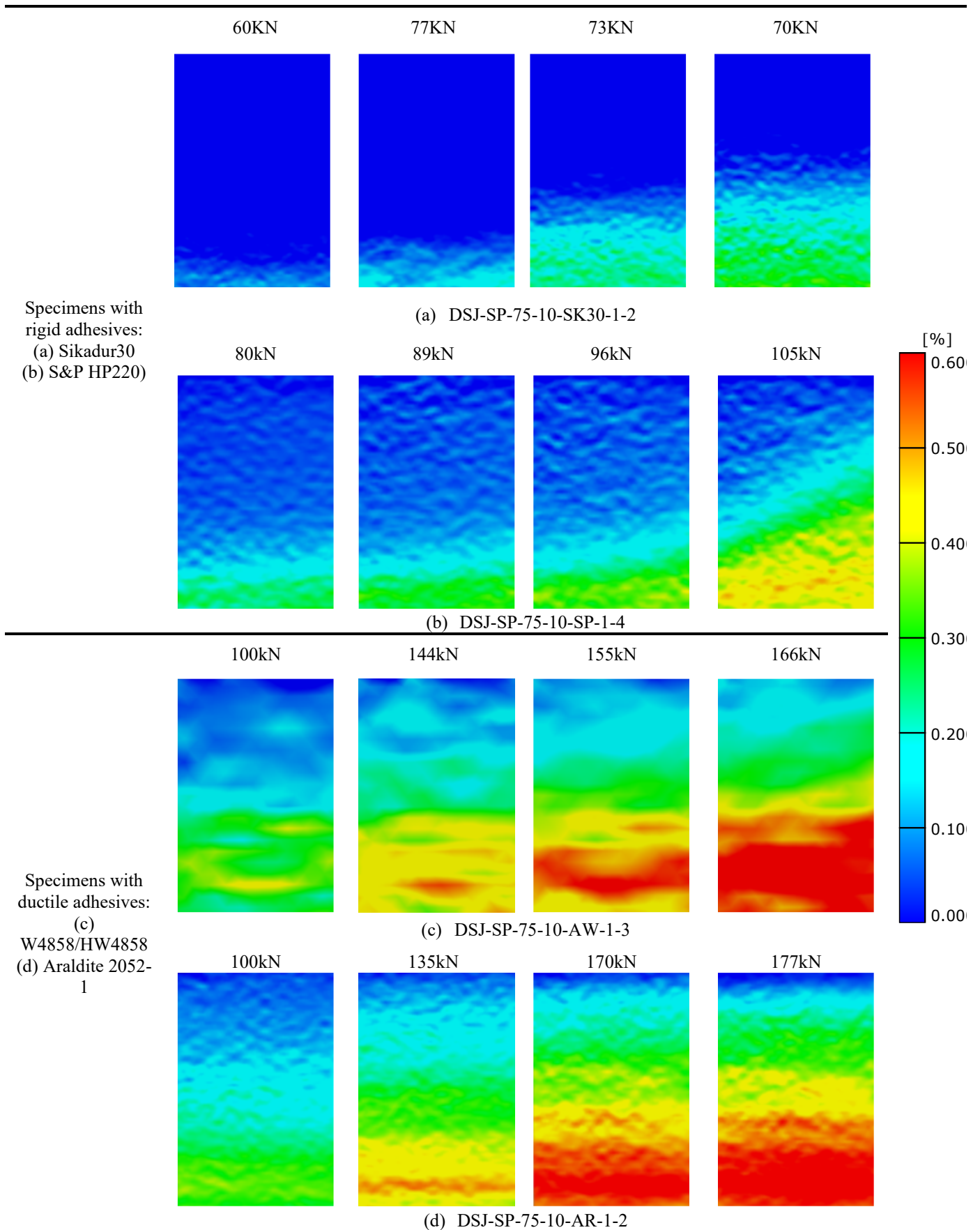


Figure 3-10: Strain fields on the CFRP back face for the double strap joints with the different adhesives (steel thickness 10mm, pultruded CFRP S&P HM(200/2000) and half CFRP length $L=75\text{mm}$).

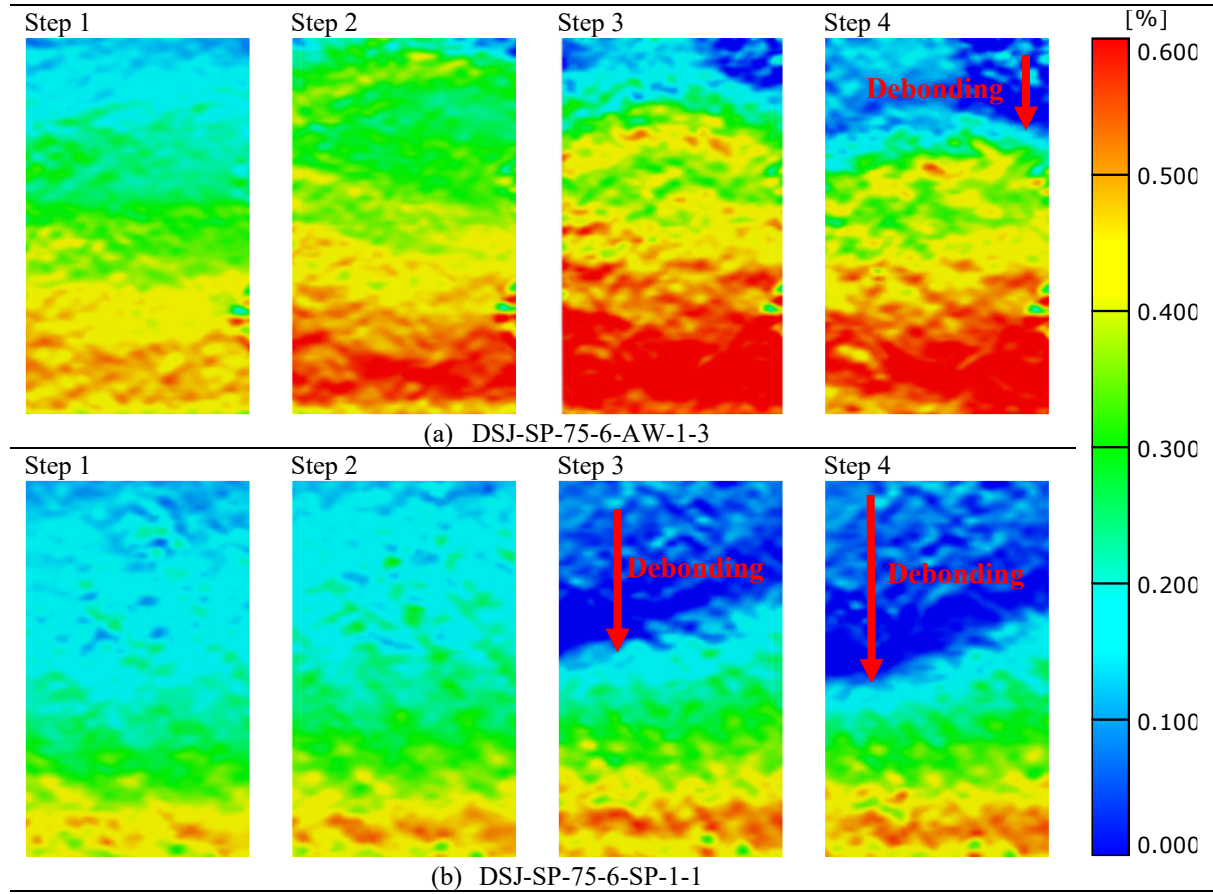


Figure 3-11: DIC contour plots on the back face of the CFRP for specimens with steel thickness = 6mm. (Note: only half of the joint is shown): (a) specimen with ductile adhesive AW4858/HW4858; (b) specimen with rigid adhesive S&P HP220.

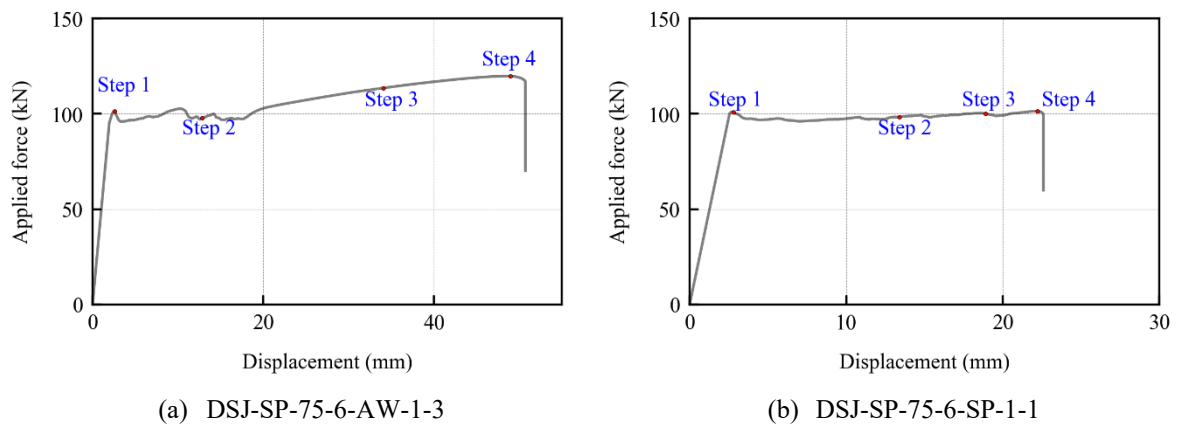


Figure 3-12: Load displacement curves showing the loading stage of each DIC contour plot shown in Figure 3-13.

3.6 Numerical analysis

A 2D Finite element model was developed in the commercial software ABAQUS to simulate the behaviour of DSJ specimens. Specimens made with steel thickness of $t_{steel} = 10mm$ and $t_{steel} = 6mm$ are modelled for comparison and verify the yielding that was observed in Figure 3-8. For the sake of simplicity and brevity, only specimens with one ductile adhesive (i.e. AW4858/HW4858) and one rigid adhesive (S&P HP220) adhesive are simulated. The others are assumed to have quite similar behavior. Note the simulation of damage in any component of the DSJ joints is beyond the scope of this study. Only the pultruded CFRP laminate are modelled in this study.

The model is shown in Figure 3-13, which consists of a steel bar, adhesive layers, CFRP plates, and testing machine grips. Half of the joint was modelled due to symmetry. Since yielding was observed in the load displacement curves of some specimens, the machine gripes were also modelled in order to capture any stress concentration zone created by the compression and the shear of the grips. The numerical model has the same geometry and dimensions of the experimental specimens used in the experiment with CFRP length equal $L_{CFRP} = 75mm$, adhesive thickness $t_{thickness} = 1mm$, CFRP thickness $t_{CFRP} = 1.4mm$ and the steel bar length $L_{steel} = 500mm$. All the model was developed using plane stress linear quadrilateral elements CPS4R available in the software library. The elastoplastic material behavior of the steel and the adhesives was modeled using isotropic model calibrated using experimental true stress-strain curves. The parameters used to model the steel behavior and the adhesives are presented in Table 3-5 and Table 3-6. A refined meshing was adopted in the numerical model with a mesh size of 0.5mm for steel bar and 0.2mm for CFRP and adhesive layer, as shown in Figure 3-14. A rigid body surface to surface tied constraints was assumed between the grips and the steel bar to ensure no slip as in the experiment. A compression force of 1MN was uniformly applied on the grips as in the experiment. The material of the grips was assumed as linear steel with 200GPa. The free edge of the grips was constrained as a rigid body to a reference point “RP1” which was in turn restrained in all degrees of freedom to ensure fixed boundary conditions. The loading was applied on the CFRP edge section as only half of the joint is modeled. The CFRP edge section was constrained as a rigid body to a reference point RP2 where a displacement loading was imposed. The

steel/adhesive interface was modeled with tie constraints where the steel surface is taken as the master surface and the adhesive surface as a slave. Similarly, for the CFRP/adhesive interface, the CFRP surface is taken as master and the adhesive surface as a slave.

Table 3-5 and Table 3-6, respectively. The CFRP has a strong unidirectional behavior following the direction of the fibers, and a weaker behavior in the other directions. Hence, it is more appropriate to model the behavior of the CFRP using an orthotropic material model. However, due to the unavailability of experimental data, the CFRP plates were assumed isotropic. This assumption is supported who Haghani [33] who investigated the effect of considering orthotropic or isotropic for similar specimens.

A refined meshing was adopted in the numerical model with a mesh size of $0.5mm$ for steel bar and $0.2mm$ for CFRP and adhesive layer, as shown in Figure 3-14. A rigid body surface to surface tied constraints was assumed between the grips and the steel bar to ensure no slip as in the experiment. A compression force of $1MN$ was uniformly applied on the grips as in the experiment. The material of the grips was assumed as linear steel with $200GPa$. The free edge of the grips was constrained as a rigid body to a reference point “RP1” which was in turn restrained in all degrees of freedom to ensure fixed boundary conditions. The loading was applied on the CFRP edge section as only half of the joint is modeled. The CFRP edge section was constrained as a rigid body to a reference point RP2 where a displacement loading was imposed. The steel/adhesive interface was modeled with tie constraints where the steel surface is taken as the master surface and the adhesive surface as a slave. Similarly, for the CFRP/adhesive interface, the CFRP surface is taken as master and the adhesive surface as a slave.

Table 3-5: Steel material elastoplastic model input parameter.

Material	Elasticity Elastic modulus (GPa)	Poisson ratio	Plasticity Yield stress (MPa)	Plastic strain
Steel ($t_{steel} = 10mm$)	200	0.3	350	0
			370	0.022
			440	0.0517
			500	0.1
Steel ($t_{steel} = 6mm$)	200	0.3	324	0
			340	0.022
			420	0.0517
			480	0.1

Table 3-6: Adhesive elastoplastic model input parameters.

Material	Elasticity Elastic modulus (GPa)	Poisson ratio	Plasticity Yield stress (MPa)	Plastic strain
AW48584/HW4858	1.32	0.35*	35.5 36	0 0.15
SP HP220	7.6	0.35*	N.A.	N.A.

* Assumed parameter.

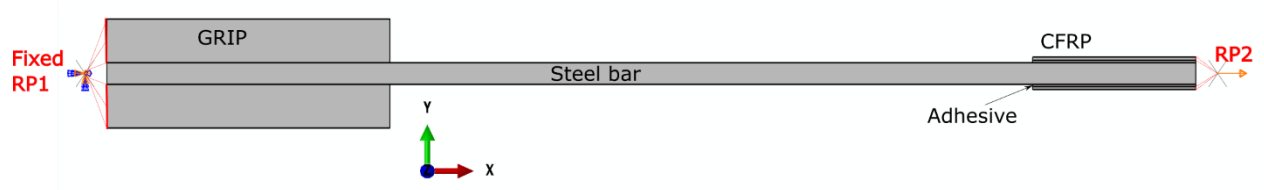


Figure 3-13: Geometry of the numerical model.

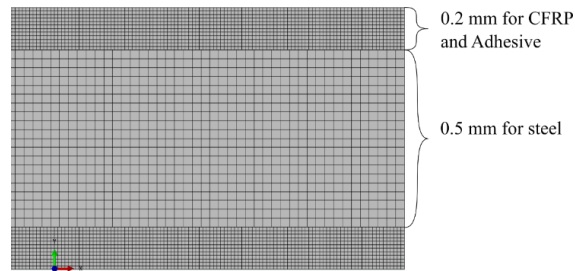
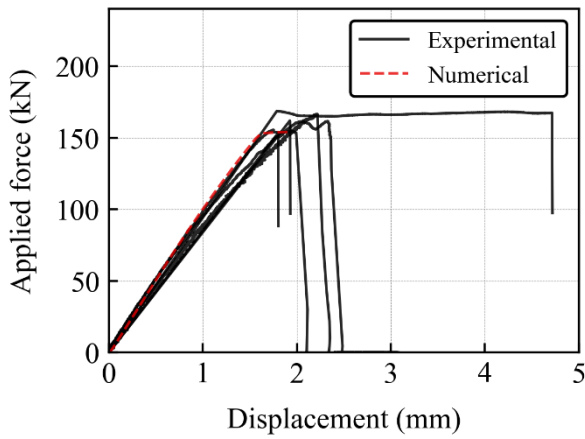


Figure 3-14: Mesh size.

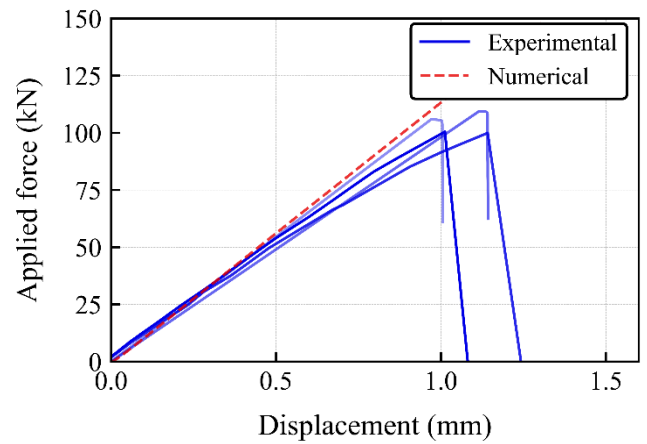
Figure 3-15 shows that the numerical results match fairly well the experimental data, which validates the experimental results shown previously. The numerical load-displacement curve of the double strap joint made with steel thickness $t_{steel} = 10\text{mm}$ and AW4858/HW4858 adhesive representative to experimental specimens “DSJ-SP-75-10-AW-1-repetition No” reach a maximum force of 152kN and then shows a plateau. At this stage of loading, maximum stress is 340MPa, which represents 97% of the yield stress. Referring to Figure 3-16, the active yield parameter indicates that at this stage the steel did not experience any plastic deformation, but the limited plastic deformation observed in the curve is due to the adhesive plastic behavior, as shown in Figure 3-16. Note that the specimen “DSJ-SP-75-10-AW-1-3” that experimentally reached a maximum load of 168kN and developed large ductile behavior, the

steel should have yielded and Likewise specimens with $t_{steel} = 6mm$, which developed large ductile behavior. Figure 3-15 (b) shows the behavior of specimen with $t_{steel} = 10mm$ and S&P HP220 adhesive experienced only a linear behavior in the steel and the adhesive before the brittle failure that occurred in the experiment. Figure 3-15 (c) and (d) show specimens with $t_{steel} = 6mm$ and both adhesives AW4858/HW4858 and S&P HP220, respectively. The numerical model could successfully capture the nonlinear behavior observed experimentally. The specimens developed mainly the significant plastic deformation in the steel bars between the grips and the CFRP patch as shown in Figure 3-16. This induced a concentration of deformation at the steel-adhesive interface at the end of the overlap, which can be clearly seen in Figure 3-17, for both specimens with the ductile adhesive AW4858/HW4858 or the brittle adhesive S&P HP220, where the ductile adhesive could develop much larger deformation.

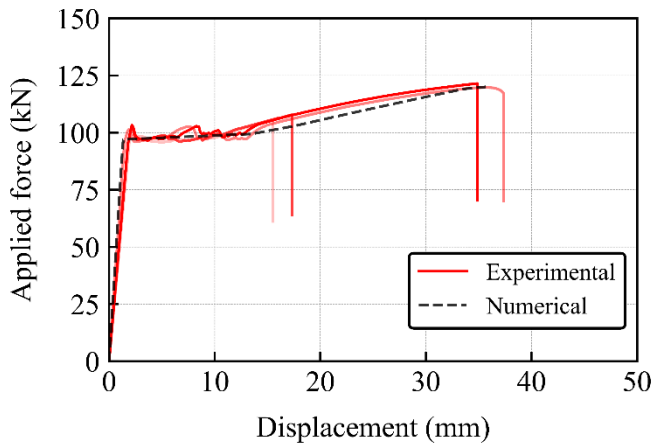
Note the behavior of specimens with $t_{steel} = 10mm$ exhibited a different behavior, as shown in Figure 3-19, where the distribution of the logarithmic strain LE11 mainly concentrate in the middle of the joint (i.e. the right side, only half of the joint is modeled). The deformation concentrates mainly near the CFRP-adhesive interface where the minimum negative deformation is observed in the adhesive layer and the large positive deformation on the CFRP side which leads to very high shear stresses. This is true for both AW4858/HW4858 and S&P HP220 as shown in detail C and detail D in Figure 3-19. Whereas near the end of the overlap (e.g. detail B Figure 3-19) the level of deformation is very low. Figure 3-18 shows that peeling deformation is also significant near the CFRP-adhesive interface for a specimen with $t_{steel} = 10mm$. The concentration of shear deformation and peeling near the CFRP-adhesive interface at the middle of the joint for specimens with $t_{steel} = 10mm$ make this region the most vulnerable for debonding.



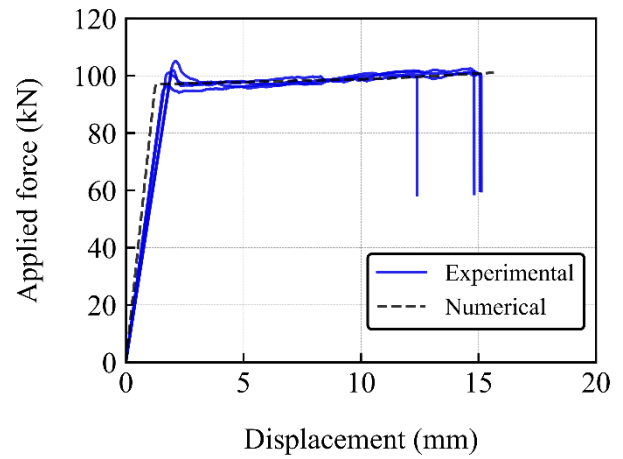
(a) DSJ-SP-75-10-AW-1-repetition No.



(b) DSJ-SP-75-10-SP-1-repetition No.



(c) DSJ-SP-75-6-AW-1-repetition No.



(d) DSJ-SP-75-6-SP-1-repetition No.

Figure 3-15: Comparison between experimental and numerical results for DSJ specimens with $t_{steel} = 10\text{mm}$ (Fig. a and b) or 6mm (Fig. c and d), AW4858/HW4858 adhesive (Fig. a and c), and S&P HP220 adhesive (Fig. b and d).

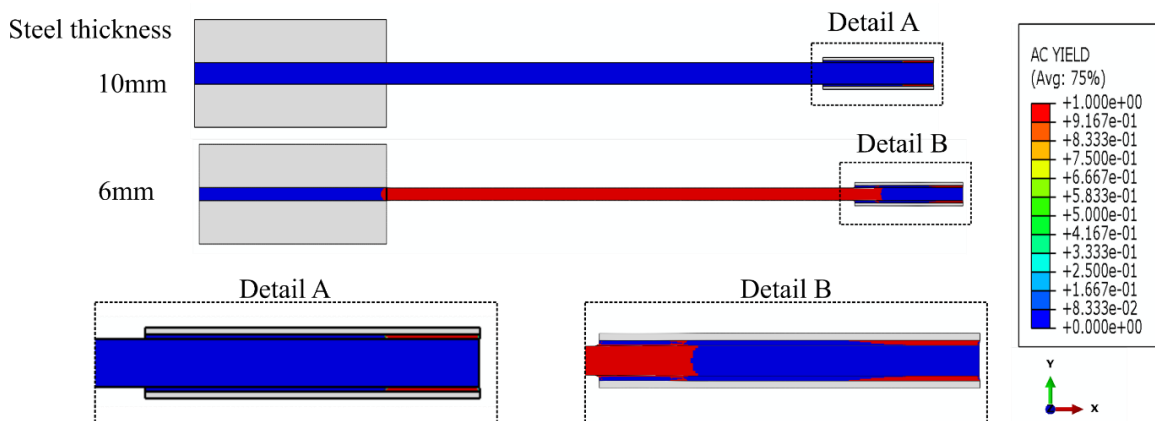
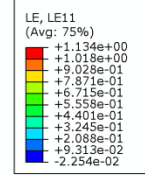


Figure 3-16: Active yield parameter for specimens with AW4858/HW4858 adhesive.

AW4858/HW4858



S&P HP220

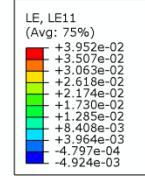


Figure 3-17: Logarithmic deformation (LE11) in the longitudinal direction(X-X) for specimens made with $t_{steel} = 6mm$.

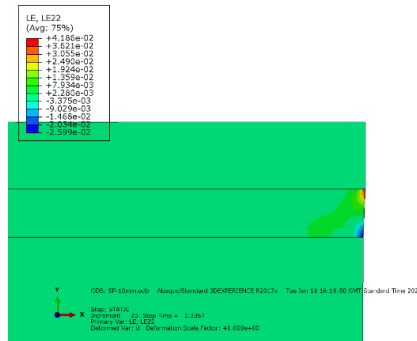
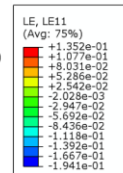
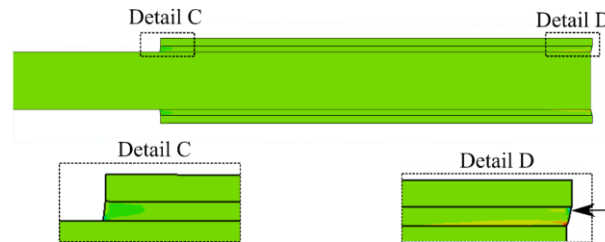


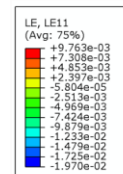
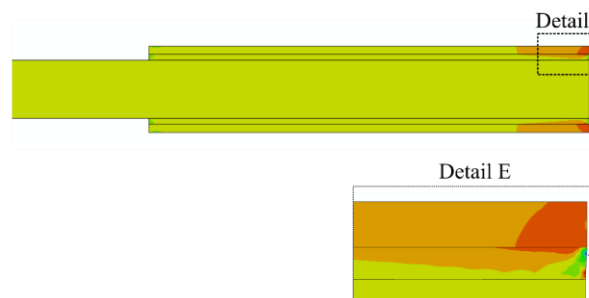
Figure 3-18: Peeling deformation LE22 (Y-Y direction) for 10mm-SP.

AW4858/HW4858



min=-1.941e-1

S&P HP220



min=-1.97e-2

Figure 3-19: Logarithmic deformation (LE11) in the longitudinal direction(X-X) for specimens made with $t_{steel} = 10mm$.

3.7 Discussion

3.7.1 Effect of the adhesive type

Ductile adhesives (i.e. AW4858/HW4858 (AW) and Araldite 5052-1 (AR)) used in this study have high yielding strength and large deformation capacity which make them extremely tougher than the brittle adhesives (i.e. SIKADUR30 (SK30) and S&P HP220 (S&P)). Figure 3-20 compares the strength of the specimens presented in Table 3-4 “control: Adhesive type”. Clearly, specimens made with ductile adhesives achieved very high strength compared to the ones made with brittle adhesives. Specimens with the brittle adhesive SK30 achieved a mean strength of $76kN$ with a standard deviation of $5.52kN$ and specimens with a slightly tougher rigid adhesive (S&P HP220) achieved a mean strength of $105kN$ with a standard deviation of $3.65kN$. Whereas specimens with ductile adhesives AW and AR reached mean strengths of $161kN$ and $171kN$ with standard deviations of $5.45kN$ and $8.6kN$, respectively. The difference of performance between ductile adhesives and rigid ones is significant with a mean strength ratio ranging from 1.53 to 2.25. As shown earlier from the DIC results, ductile adhesives promote a more uniform distribution of deformation in the bonded joint which reduces the stress concentration.

It was also observed in section 3.2 that the adhesive type influenced the failure mode. The failure in the CFRP/steel bonded joints is a crack that initiates and propagates in the component that satisfies the Griffith energy criterion $G > G_c$, where G is the energy demand on the material and G_c is a material property called critical strain energy release rate. The delamination failure mode occurs due to a crack in the CFRP matrix. For specimens presented in Table 4 “control: Adhesive type”, the failure modes were either cohesive or CFRP delamination. When the adhesive is tougher than the CFRP matrix, the failure mode occurs in the CFRP and vice versa. This can be clearly observed in these experimental results were for lower toughness adhesive (i.e. SK30), cohesive failure governed the failure mode, whereas, for tough ductile adhesives such as AW and AR, CFRP delamination was the failure mode. For specimens with S&P HP220 which is moderately tougher than SK30, a combination of CFRP delamination and limited cohesive failure governed the failure mode.

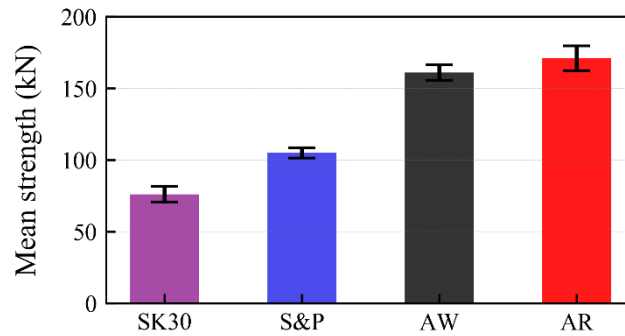
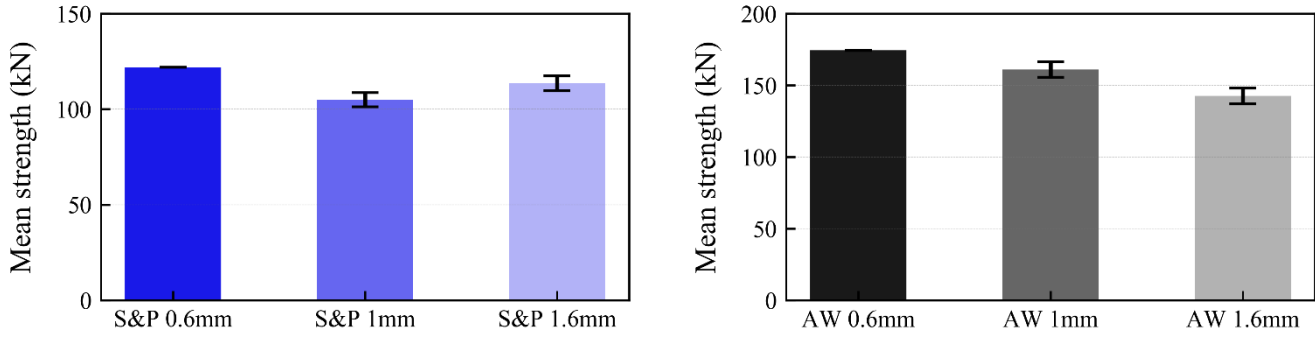


Figure 3-20: Effect of adhesive type on the DSJ mean strength.

3.7.2 Effect of adhesive thickness

The effect of the adhesive thickness on the strength of adhesive joints is a complex and open issue. The adhesive thickness may influence the stress state in the adhesive layer and enlarge the distribution of shear stresses. However, the eccentricity generated by thickness gives rise to higher peeling stress. Besides, larger voids are likely to be present in thicker adhesive bond lines. From a fracture mechanic point of view, researchers gave conflicting conclusions, while some researchers [34] suggested that with decreasing adhesive thickness, the fracture energy reduces significantly, others [35] observed that adhesive thickness influences the fracture process zone shape (FPZ) causing the adhesive joint fracture to behave differently where thinner adhesive bond lines lead to a stretched thin FPZ, delaying the crack propagation. From Table 4, specimens with S&P HP220 adhesive with 1mm thickness failed under delamination dominant failure mode whereas specimens with 0.6mm failed under cohesive dominant failure mode. This can be seen, in Figure 3-8 (c) and (d) which illustrate a specimen with a thin layer (0.6mm) that failed under cohesive dominant failure mode and a thicker layer (1mm) under delamination dominant failure mode. In this study, specimens made with SP and AW adhesives were tested with three different thicknesses (i.e. 0.6mm, 1mm, 1.6mm) for comparison, as shown in Table 4 “control: Adhesive thickness”. Figure 3-21 (a) shows the variation of mean strength for specimens made with S&P HP220 adhesive. As can be seen, the strength fluctuates, and the results are not conclusive. This has been also observed in the experiments of Yu et al. [7] for specimens with similar brittle adhesive. Figure 3-21 (b) shows that the joint strength decreases with the increase of the adhesive thickness.



(a) specimens with S&P HP220 adhesive

(b) specimens with AW adhesive

Figure 3-21: Effect of adhesive thickness on the mean strength of DSJ specimens: (a) DSJ-SP-75-10-SP-“Adhesive thickness”-“Repetition No.”; (b) DSJ-SP-75-10-AW-“Adhesive thickness”-“Repetition No.”. Note: thickness of the adhesive (i.e. 0.6mm, 1mm, or 1.6mm) of each joint type is presented in the bar charts.

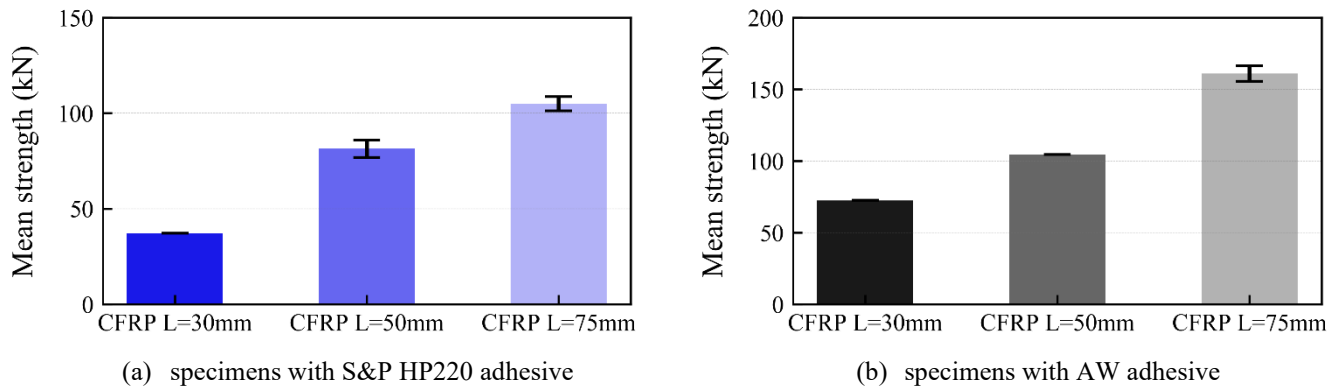
3.7.3 Effect of CFRP modulus

Two types of CFRP were used in this study, namely: normal modulus CFRP ($E = 205GPa$) and high modulus CFRP ($E = 640GPa$). Joints with both could achieve very high strength when bonded with ductile tough adhesives AW. Specimens with high modulus CFRP and $t_{steel} = 6mm$ achieved the yield strength and developed a ductile behavior as for specimens with the pultruded CFRP. Similarly, for the two tested specimens with high modulus CFRP and $t_{steel} = 10mm$, a very high strength could be achieved with values of 161.3kN and 173.3kN which is quite similar to the ones made with the normal modulus where the strength varied between 155.5kN and 168kN. However, the fracture behavior of the joints with two types of CFRP is different. For joints with high modulus CFRP, a rupture of CFRP in the middle of the joint occurred which is mainly due to the low deformation capacity of the high modulus CFRP. Whereas joints with normal modulus CFRP, delamination failure occurred.

3.7.4 Effect of CFRP length

Figure 3-22 (a) and (b) illustrate the variation of the mean strength with respect to the CFRP length for specimens with S&P HP220 adhesive and AW adhesive, respectively. Clearly, the mean strength increases with the increase in the CFRP length. Note a maximum length of $L = 75mm$ was used in this study. Longer CFRP may lead to an increase of strength until reaching a limit where the no further increase can be reached. Usually, the CFRP length corresponding to this limit is called the effective length in the literature [10]. However, the determination of the effective bond length is beyond the scope

of this study. The CFRP length considered here are intended to represent a repair patch in crack structural details such as in the case of connections where limited space is available for longer CFRP.



(a) specimens with S&P HP220 adhesive

(b) specimens with AW adhesive

Figure 3-22: Effect of CFRP length on the mean strength of DSJ specimens: (a) DSJ-SP-“CFRP length”-10-SP-1-“Repetition No.”; (b) DSJ-SP-“CFRP length”-10-AW-1-“Repetition No.”.

3.7.5 Effect of steel thickness

DSJ Specimens with $t_{steel} = 6mm$ and $t_{steel} = 10mm$ were tested for comparison of the behavior. Due the lower yield strength of this steel bars ($t_{steel} = 6mm$), the DSJ specimens developed large plastic deformation before failure. Numerical results revealed that the plastic deformation occurred mainly in the steel bars between the CFRP patch and the machine grips used in the experiment. The DIC results showed that yield of the steel bars influence the distribution of deformation on the CFRP backface and the failure. For specimens made with the rigid adhesive SP HP220 “DSJ-SP-75-6-SP-1-repetition No.” the failure mode was mainly in the steel-adhesive interface due the large plastic deformation occurred in the steel. Specimens made with the ductile adhesive did not failure in the steel-adhesive interface due to the high ductility of the adhesive. Most specimens with $t_{steel} = 10mm$ did not develop plastic deformation in the steel and failure generated from the gap region in the middle of the joint.

Although steel yielding is usually avoided in the experimental studies in the literature [7] by using thick rigid steel blocks instead if using double strap joints with relatively thin bars. However, the steel thickness influence significantly the behavior of adhesively bonded joint, which can change completely the distribution of stress and the failure mode. Therefore, it is important to use steel elements with thicknesses representative to practical situations. For example, $t_{steel} = 6 - 10mm$ can be found in

most old metallic bridges made with riveted technology, which was the main reason for the choice of the thicknesses used in this study.

3.7.6 Surface treatment

The surface treatment is a very important step for the successful bonding of the adhesive joint. As stated previously, sand blasting with alumina dioxide grain size between 212-300, used in this study, is the most recommended method [12]. However, for acrylic based structural adhesive Araldite 2052-1, the manufacturer claims in the datasheet that no or little surface preparation is required for perfect bonding. So in this study, four DSJ joints were tested with Araldite 2052-1, where two of the them the steel was sand blasted (i.e. DSJ-SP-75-10-AR-1-“repetition 1 and 2”) and two were only polished using angle grinder with sand-paper grit 80 (i.e. DSJ-SP-75-10-AR-1-“repetition 3 and 4”). As shown in Table 4, the results revealed that the specimens polished only with sand paper achieved very high strength of 175.1kN and 175.3kN and specimens with sand blasting achieved 156.2kN and 177.5kN. It can be concluded that the adhesive Araldite 2052-1 requires very low surface preparation which can be very useful in practical situations.

3.7.7 Failure mode

Usually, it is assumed that a well-designed adhesive joint will fail cohesively. This might be true in the case of very tough adherends such as in metal-metal adhesively bonded joints. However, using extremely tough adhesives as the once used in this study render the cohesive failure not possible in a CFRP joint. The pultruded CFRP available in the market are usually made with vinyl ester adhesives which have lower toughness than the extremely tough ductile adhesives. The increase of the CFRP interlaminar toughness maybe increased by using tougher adhesive in the impregnation however, the concentration of deformation in the adhesive-CFRP interface at the middle of the joint as shown in Figure 3-19 will always favourite a delamination failure since the first CFRP layers in the laminate will be subjected to high shear stress and peeling stress.

Specimens with lower steel thickness are likely to experience steel yielding than failure in the adhesive-steel interface starting from the end of the overlap. Note this failure mode is not a premature failure due

inadequate surface treatment. The authors believe that this the most suitable failure mode for adhesive joints because the joint develop very large ductility in the steel before failure and more importantly this means that the CFRP adhesive joint is stronger than the yielding strength of the parent material which is steel.

3.8 Joint strength efficiency for steel/CFRP adhesively bonded joints

As shown throughout this study, CFRP-steel adhesive joints achieve very high strength when bonded with ductile tough adhesive. Some specimens with $t_{steel} = 10mm$ and all specimens with $t_{steel} = 6mm$ developed a ductile behavior in the steel part. As stated previously, this is the most suitable failure mode for CFRP/steel adhesive joints since it means that the adhesive joint is stronger than the yield strength of the parent steel structural detail. Besides, the yield strength of the parent material (steel) can be used as a design parameter of the adhesively bonded joints. Similarly to the joint efficiency used for welded joints which refers to the strength ratio of the joint with respect to the strength of the base metal. A Joint Efficiency of 100% indicates that the joint has the same strength of the base metal and can be used confidently in practical situations. This design criterion can be significantly simplifies the design procedures for CFRP/steel adhesively bonded joint, without the need to carry out cumbersome design checks on the fracture of the adhesive layer or the CFRP, which is very complex. So further development of design rules based joint efficiency are higher recommend in future research.

3.9 Conclusion

This study focused on the characterization of CFRP/Steel adhesively bonded joints. For this purpose, experimental and numerical analysis were conducted on CFRP/steel double strap joints including several variants such as the adhesive type, steel thickness, CFRP modulus, CFRP length, adhesive thickness, and surface preparation. The results revealed that:

- Steel/CFRP joints with ductile tough adhesive achieve much high strength than the ones made with rigid adhesives commonly used in construction practice. The failure modes for specimens made with rigid adhesive is mainly a combination of cohesive and CFRP delamination failure, whereas the joints made with ductile adhesive is only CFRP delamination. The CFRP/steel

joints with ductile adhesives cannot fail under cohesive mode due to the high toughness of the adhesive compared.

- When steel yielding occurs in the part outside of the joint region, the joint develop a ductile behavior and the distribution of stress in the adhesive bond line changes and stress concentration become significant at the end of the overlap in the steel/adhesive interface. Joints with brittle adhesive, the failure mode that governs the behavior is adhesive failure at the steel/adhesive interface, whereas ductile adhesive can sustain the large plastic induced by the steel part and the joints mainly fail under CFRP delamination.
- For joints with ductile adhesive, an increase of adhesive thickness was associated with a decrease of strength. Whereas specimens with rigid adhesive the results fluctuated and the influence could not be observed.
- Joints with both normal modulus CFRP and high modulus CFRP achieved very high strength when bonded with a ductile adhesive. DSJ specimens with high modulus CFRP, failed under CFRP rupture in the middle and joints with normal modulus CFRP, exhibited CFRP delamination mode.
- DSJ specimens with the acrylic based adhesive that received low quality of surface treatment (sandpaper) achieved very high strength similar to the ones prepared with sand blasting.
- A design approach similar to the joint efficiency methodology used for welded connection is suitable for adhesively bonded joints and is strongly recommended.

3.10 Reference list:

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CHAPTER 4: Cost effective bi-adhesive CFRP/Steel joint for the construction industry

4 Cost effective bi-adhesive CFRP/Steel joint for the construction industry

Abstract

This chapter presents an experimental and numerical study on the behavior of adhesively bonded Steel/CFRP joints. As it was shown in chapter 3, ductile tough adhesive provide very joint high performance compared to brittle adhesives used in the construction industry. However, the cost of tough adhesives may question the feasibility of this repair solution. Therefore, in this study a cost effective CFRP/steel joint is proposed using bi-adhesive technique, which consist of using only small amount of the tough (expensive) adhesive and large amount of the brittle adhesive commonly used in the construction industry. The experimental and numerical results showed that the proposed bi-adhesive joint achieve high strength and promotes a more uniform stress distribution.

4.1 Introduction

The mechanical properties of the adhesive have a significant influence on the performance of the steel/CFRP adhesive joints. Nowadays, extremely tough adhesives can be found in the market. As it was shown in chapter 3, This type of adhesive should provide very high performance suitable for metallic bridge repairs, however, they can be significantly more expensive than brittle adhesives used in the construction industry (at least 10 more expensive). In civil engineering structures, large amount of adhesive is required due to the size of the structural elements. Therefore, the use tough adhesives might be economically unfeasible. The development of more economical solutions while maintaining the same level of performance of tough adhesives joints needs to be developed.

Several strengthening technics of adhesive joints have been adopted in the literature such as tapering the overlap and/or using large fillets of adhesive [1]–[3], reinforcement of the adhesive layer with metallic or organic fibres [4], [5], and mixed adhesive joints [6]. Mixed adhesive joint or also called bi-adhesive joints have been firstly recognized by Hart-smith [7] for low and high temperature applications to extend the temperature range of the adhesive joint. Other researchers investigated bi-adhesive joints to improve the joint strength in relation [6], [8]–[11]. This technique consists of using a stiff and strong adhesive in the middle of the overlap and a very flexible and ductile adhesive at the ends of the overlap

to relieve the high peeling stress at the ends of the overlap and promotes a more uniform strain distribution. Past experimental results [6], [8]–[11] revealed that bi-adhesive joints achieve higher strength than brittle adhesive alone. However, most the existing studies were developed for the aeronautical or automotive industry where the joint geometry such as the overlap length and the adhesive thickness are not representative to civil engineering structures. Besides, most studies used ductile adhesive that exhibits greater strain capacity but low bulk strength due to the less cross-linked polymeric network [10], which compromises the joint strength compared to the ductile adhesive.

This chapter presents an experimental investigation on a modified geometry bi-adhesive joints made mainly with a brittle adhesive commonly used in construction practice and an extremely tough adhesive that combines high strength and ductility. This study compares the performance of the brittle and extremely tough adhesive joints and propose a more economical adhesive joints with improved strength since the cost of the use tough adhesive alone may question feasibility of the repair. First, bulk adhesive coupon tensile tests are conducted to characterise the basic mechanical properties of the adhesives such as tensile strength, young modulus, deformation capacity, and toughness. Then, adhesively bonded CFRP/steel double strap joints, representative to a composite patch repair over the cracked region [12], [13], are tested under static tensile loading. A comparison between the performance of the joints with brittle and tough adhesives is provided. Then, a cost effective and improved performance adhesive joint is proposed using bi-adhesive technology.

4.2 Experimental program

4.2.1 Adhesive basic mechanical properties

Three different adhesives have been used to bond the CFRP and the steel plates, where two of them are brittle adhesives (i.e. SIKADUR 30 (SK30) and SP HP 220 (SP)) commonly used in the construction industry and one extremely tough adhesive (Araldite AW4858/HW4858 (AW)) suitable for bonding a wide variety of metals, and especially designed for bonding composites [14]. Coupon tests were conducted according to ASTM D638-14 [15] with specimens' geometry category 5 which is recommended for comparison between different adhesives. The specimens were made in acrylic moulds and cured at room temperature for at least 7 days. The specimen dimensions and test set up are

shown in Figure 4-1. The tests were conducted in the MTS 831-02 Elastomer test system with a speed of 1mm/min. The strain measurements were acquired using extensometers. The basic average material properties of the adhesives are presented in Table 1. Figure 4-2 illustrates the stress-strain curves of the three different adhesives. The adhesive SK30 reach high strength above 35 KN and high stiffness but very low strain capacity up to 0.3%. The adhesive SP has a lower stiffness than SK30 and reaches roughly similar strength but with doubled deformation capacity. The adhesive AW has lower young modulus than the two brittle adhesives but reach similar strength around 35MPa and significantly higher elongation at break with an average factor of 64 and 48 compared to SK30 and SP, respectively. As expected, the toughness measured as the area under the stress-strain curve is extremely higher for the tough adhesive than the two other brittle ones.

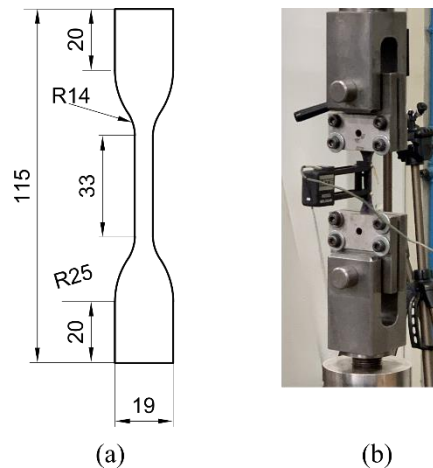


Figure 4-1: Specimens' dimensions (a) and testing set up (b).

Table 4-1: Average adhesives basic mechanical properties.

Adhesive	E (N/mm ²)	Stress Max (N/mm ²)	Deformation capacity (%)	Toughness (N/mm ²)
Sikadur 30	12915	35.03	0.28	0.058
S&P HP220	887	33.16	0.54	0.108
AW4858/HW4858	1770	34.9	13.6	3.9

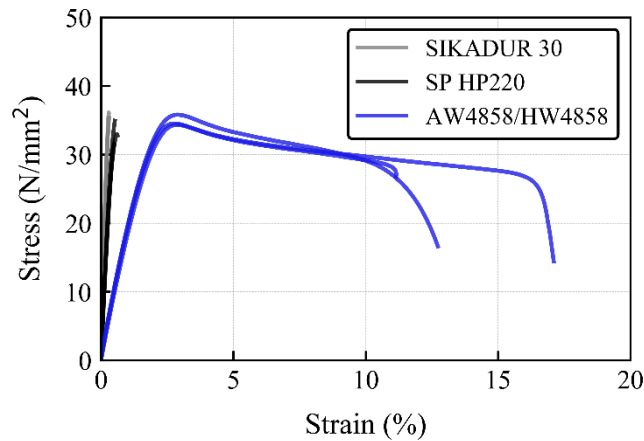


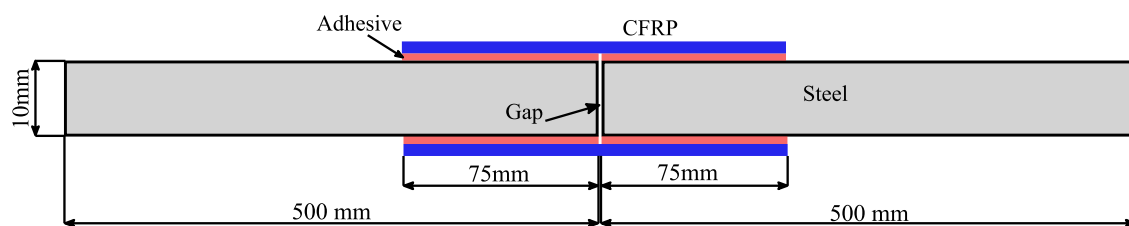
Figure 4-2: Dog-bone test stress-strain curves.

4.2.2 CFRP/steel double strap joints

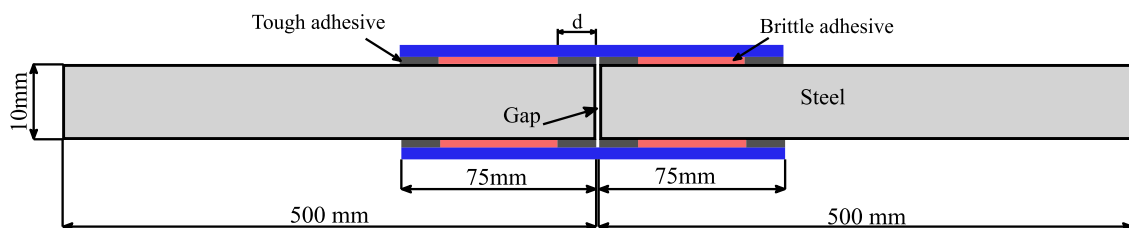
Firstly, CFRP/steel double strap joints bonded with single adhesive (i.e. SK30, SP HP220 and AW) were tested for comparison between the behavior of brittle and tough adhesives. Each set of specimens consisted of at least 5 repeated tests. A schematic drawing of the specimens with single adhesive double strap joints is shown in Figure 4-3 (a). The double strap joint is constituted by two separate steel bars joint together with a bonded CFRP patch in both sides. Unidirectional pultruded CFRP laminates (S&P C-Laminate HM (200/2000)) with a thickness of 1.4 mm, 50mm wide and a young modulus of 205GPa were used. The steel bars were made of European hot rolled carbon steel S275 with 50 mm width, 10mm thickness, and 500 mm length. The measured yields stress, young modulus, and ultimate stress are 350MPa, 200GPa, and 500MPa, respectively. The adhesive thickness was taken equal to 1mm for all the specimens.

Then, bi-adhesive joints using a combination of tough (ductile) and brittle adhesives were manufactured and tested. The details on bi-adhesive layer are shown in Figure 4-3 (b), where two different adhesives are used and the intended length covered by the tough adhesive is distinguished by a letter “d”, which is equal to 10mm. Note the configuration proposed in this study is different than existing studies. Previous research mainly applied the ductile adhesive only in the ends of the overlap to reduce the peeling stress. Inhere, the ductile adhesive is applied in the ends and in the middle (at the joint level). The rationale behind is that the double strap joints experience high shear stress concentration in the

middle of the joint and since the ductile adhesive used inhere has a very high shear strength and ductility, it is assumed that it will contribute significantly to the resistance of crack initiation in the middle of the joint as well as at the ends of the overlap. The bi-adhesive specimen is mainly made of the brittle adhesive S&P HP220 and the tough adhesive (AW4858). The choice of the brittle adhesive S&P HP220 instead of SK30 was taken based on the experimental results of single adhesive DSJ joints (shown in section 3.4) where specimens with S&P HP220 showed a higher performance than the once with SK30. For single adhesive joints, the specimens are distinguished based on the adhesive used, where the nomenclature is presented in the form “*DSJ-Adhesive type*”. For example, a double strap joint with adhesive Sikadure 30 is named as “DSJ-SK30”. Whereas, bi-adhesive joints are named “DSJ-Bi” since there are no variants.



(a) Single adhesive double strap joint



(b) Bi-adhesive double strap joint

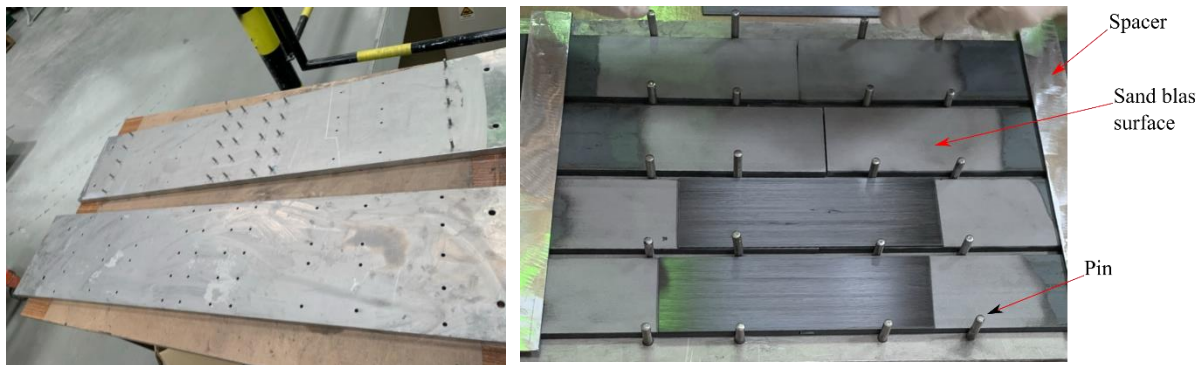
Figure 4-3: Schematic view of a double strap joint.

4.2.3 Specimens' manufacture

The preparation of the specimens involved mainly two steps: (a) surface preparation of the steel and CFRP and (b) application of the adhesive between CFRP and steel in the mould. The surface preparation is a key step for the successful bonding of structural adhesives [16]. A proper surface treatment provides

a rough surface favoring mechanical interlocking between the adherends and the adhesive to further increase the adhesion strength. The steel plates were sand blasted using alumina dioxide sand F60 with grain size between 212-300 micro, as recommended in [17], to avoid adhesion failure in the steel-adhesive interface which usually occurs due to inadequate surface preparation. Then, the specimens were cleaned with compressed air and acetone to remove any particles left from the sandblasting operation and to have a chemically active surface ready for bonding. The CFRP surface was gently polished in the fiber direction with sandpaper grit 180. Then, the CFRP plates were cleaned with compressed air and acetone.

The specimens were made in aluminium moulds that have been machined specifically for these specimens. The moulds are made of two thick flat aluminium plates of 25 mm thickness, 1300 mm long and 250 mm width. Each mould can accommodate 4 specimens in the longitudinal direction separated by pins, as shown in Figure 4-4. The main role of the pins is to ensure that the CFRP and the steel bars are well aligned. Aluminium spacers of 2.4mm were used to maintain a constant 1mm adhesive thickness (i.e. 1mm). After closing the mould the upper plate will apply a uniform pressure on the specimens. Then, the specimens were left to cure for at least 7 days at room temperature.



(a) Aluminium mould covered with unmoulding agent

(b) Demonstration of the alignment of the specimens.

Figure 4-4: The aluminium Mould used for manufacturing the double strap joints.

4.2.4 Experimental set-up

The double strap joint specimens were tested in ESH testing machine in the materials lab at Faculty of Engineering University of Porto (FEUP), as shown in Figure 4-5. The specimens were subjected to static tensile loading until complete failure. The loading speed was 1 mm/minute. The strain fields on

the back face of the CFRP plates and on the lateral face of the joint were captured using 2D digital image correlation (DIC). A random speckle pattern with black background and white spots was applied on the back face of the CFRP. Note, a maximum of 100mm lens imaging was available so the lenses were controlled to achieve a sharp focus only on half of the joint.



Figure 4-5: Experimental set up.

4.3 Load displacement curves

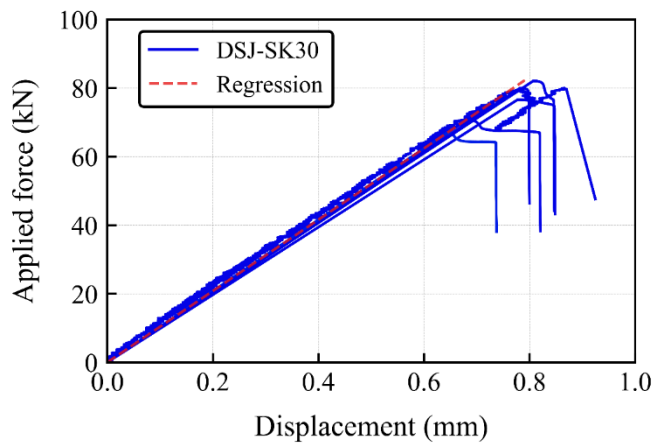
Figure 4-6 presents the load displacement curves of DSJ specimens bonded with CFRP. For specimens with SK30 shown in Figure 4-6 (a), the strength reached a value between 65-85 KN followed by a moderate flat plateau in most cases. The flat plateau has been reported in other studies in the literature [18] when cohesive failure is observed for joints with Sikadur 30. The maximum strength is associated with crack initiation in the middle of the joint and the flat plateau is attributed to the crack propagation which depends on the length of the bonded area. Note the voids are inevitable in adhesive joints and their volume and location have a significant influence on the crack initiation and propagation in the adhesive joint, which justifies the variability in the strength the repeated tests. Figure 4-6 (b) presents the load displacement curves of DSJ specimens bonded with S&P HP220. The strength fluctuates around 100 to 120KN, followed by an abrupt failure due to delamination failure mode. Figure 4-6 (c)

illustrates the load-displacement curves for specimens made with the tough adhesive AW4858/HW4858. The maximum load reached by the specimen is between 150KN-170KN, which corresponds to the yield strength of the steel bar obtained experimentally. The specimens with AW4858/HW4858 failed under CFRP delamination, which is known as a brittle failure, the yielding plateau observed in the specimen that exhibited a long flat plateau is attributed to plastic deformation of the steel bars since the yield strength was reached. Whereas the behavior of the others that showed limited ductility will be verified numerically in the subsequent section. The bi-adhesive specimens reach maximum force of 174 KN. There is a clear increase of strength for bi-adhesive joints compared to the brittle adhesive joint. Therefore, the configuration of the bi-adhesive joint can be judiciously used to increase the joint strength compared to the single brittle adhesive joint while maintaining a relatively lower cost.

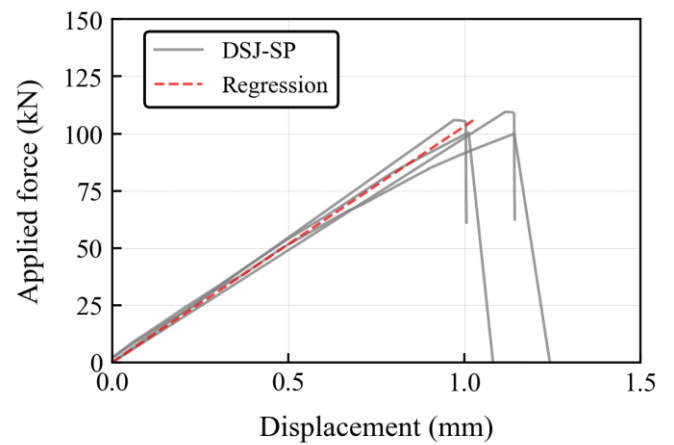
4.4 Failure modes

The failure modes and the inspection of the fracture surface provide important information on the behavior of the adhesive joint. According to Zhao and Zhang [19] and Majidi et al. [20] the possible mode of failures in the steel/CFRP joint are the followings: Adhesive failure at the adherends-adhesive interface caused usually by inadequate surface treatment [16], cohesive failure in the adhesive layer, CFRP rupture that occurs for high modulus CFRP due to its limited deformation capacity [19], CFRP delamination, and steel yielding which is unlikely to occurs as reported by Yu et al. [18]. Figure 4-7 (a) illustrates two typical DSJ-SK30 specimens, where the failure mode is a combination of cohesive failure with CFRP delamination which occurred in at the free ends of the overlap in the region of high peeling stress, similar observations have been reported in [18] for specimens with SK30. Figure 4-7 (b) illustrates typical failure mode for DSJ-SP specimens which is mainly CFRP delamination with a combination of limited cohesive failure. Figure 4-7 (c) illustrates a typical DSJ specimen with AW adhesive, where the failure mode is CFRP delamination. For bi-adhesive joints, the main failure mode is CFRP delamination with limited cohesive failure in the region of the S&P reinforcement, as shown in Figure 4-7 (d).

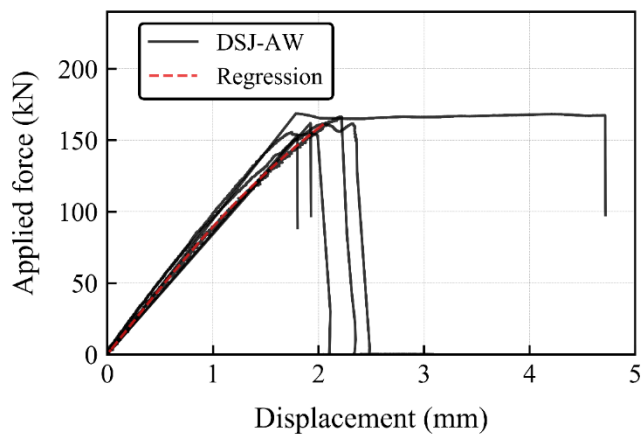
The failure in the CFRP/steel bonded joints is a crack that initiates and propagate in the component that satisfies the Griffith energy criterion $G > G_c$, where G is the energy demand on the material and G_c is a material property called critical strain energy release rate. The delamination failure mode occurs due to the a in the CFRP matrix. The failure mode observed in this study were either cohesive or CFRP delamination. When the adhesive is tougher than the CFRP matrix, the failure mode occurs in the CFRP and vice versa. This can be clearly observed in these experimental results where for lower toughness adhesive (i.e. SK30), cohesive failure governed the failure mode, whereas for tough adhesive such as AW4858/HW4858, CFRP delamination was the failure mode. For specimens with S&P HP220 which is moderately tougher than SK30, a combination of CFRP delamination and limited cohesive failure governed the failure mode.



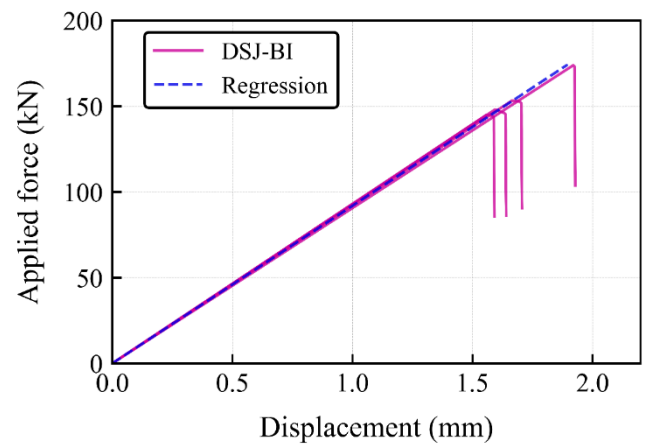
(a) DSJ with SIKADUR 30



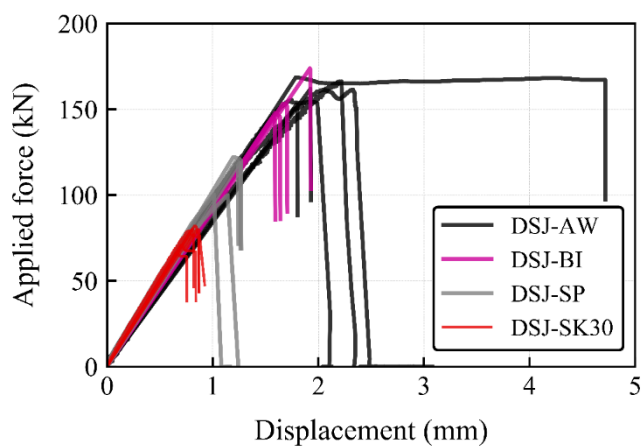
(b) DSJ with SP-HP220



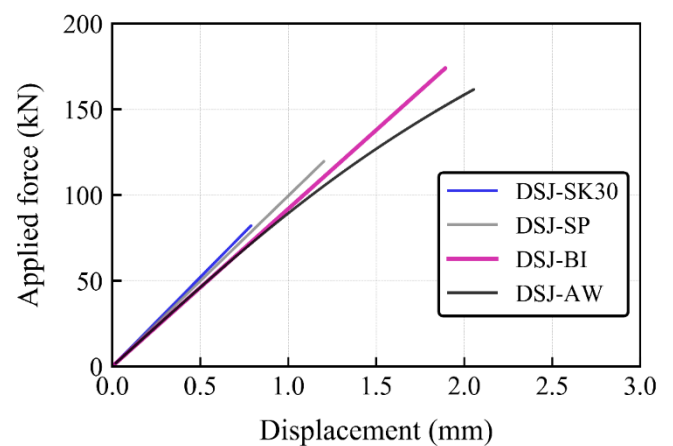
(c) DSJ with AW4858/HW4858



(d) DSJ with bi-adhesive



(e) Load displacement comparison



(f) Stiffness comparison

Figure 4-6: DSJ specimens load displacement curves.



Figure 4-7: Typical failure mode for DSJ specimens: (a) SK30 adhesive, (b) AW4858/HW4858, (c) SP, and bi-adhesive.

4.5 Strain distribution

Figure 4-11 shows the strain fields on the back face of the CFRP captured using DIC on specimens with the brittle adhesive (SP HP220), the tough (AW4858/HW4858), and mixed adhesive (SP HP220 + AW4858/HW4858). For the specimen with SIKADUR 30, the contour plots are shown in Chapter 3 where the results showed large scatter. The specimen with SP HP220 adhesive show the deformation concentrate in the middle of the joint. At higher loading levels, the strains are not distributed uniformly which may indicate the presence of a weaker zone due to manufacturing imperfections such as voids. The specimens with the AW4858/HW4858 exhibit deformation at almost all the face of the CFRP before failure. The specimens with bi-adhesive shows the tough adhesive bring through the development of a more uniform strain distribution and make use of larger area of the CFRP which leads to an improved behavior compared to the brittle specimens.

The contour plots presented in Figure 4-11 provides a good insight on the behavior of the adhesive joint. However, it is important to define the average strain-applied force curves that can be used for development of design procedure. The noise is an inherent aspect of the DIC technique which can be clearly seen in Figure 4-11. In this study, the strain measurement was taken from data points spaced by 5mm between each other in the horizontal and vertical direction, as show in Figure 4-8. Where each row would represent the behavior at a given distance from the middle of the joint. Figure 4-9 shows, the strain data at the joint level. The row data shows significant noise within each data point. In order to reduce the in-line fluctuations, the data was smoothed giving the discrete linear convolution of two one-dimensional sequences. Then, the average was calculated based on the smoothed data. Note the inline fluctuation are the noise from the DIC measurements but the differences between the smoothed lines in the same row is assumed to be due the different behavior between each point, as obviously shown in Figure 4-11 for specimen DSJ-SPHP220. In Figure 4-10 (a), the normalized strain (i.e. measured strain/mean strain) show high variability at loading lower than 60kN then the data tend to converge toward 1. Figure 4-10 (b) shows the standard deviation of the normalized strain, where it shows that there is a high variation at lower loading levels and then reduce to 0.13. This indicates that use of DIC technique at low strain levels, as in the case of specimens with SK30, should not provide

accurate results due to the significant noise. The mean strain versus the applied loading curves at each row are plotted for the DSJ specimens with SP HP220, AW4858/HW4858, and bi-adhesive Figure 4-12. Clearly, the specimens with tough and bi-adhesive exhibit larger deformation levels compared to specimens with brittle adhesive SP HP220.

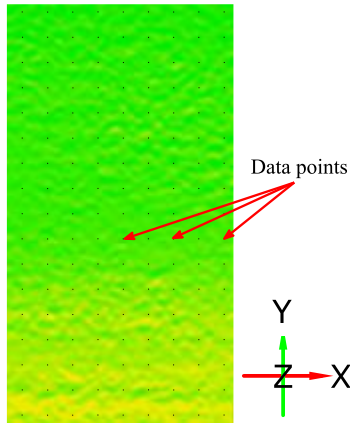


Figure 4-8: measurement data points.

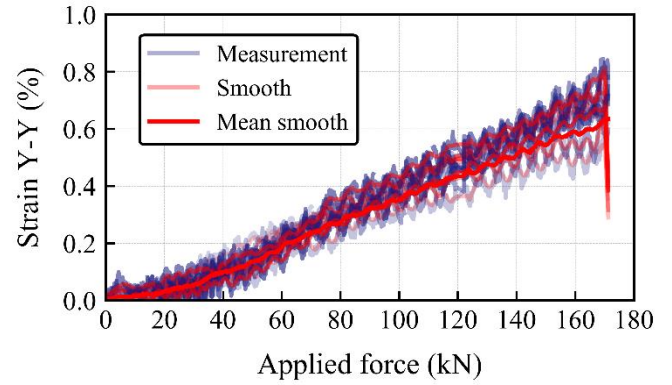
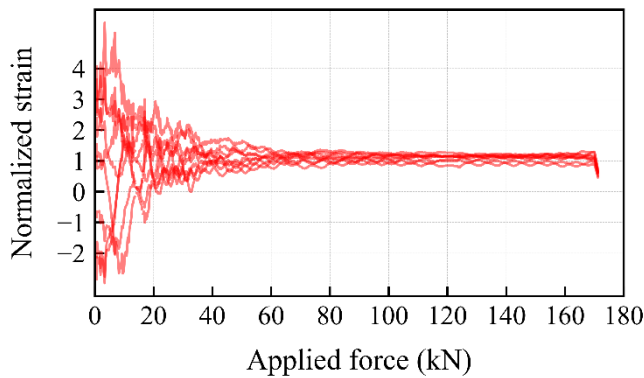
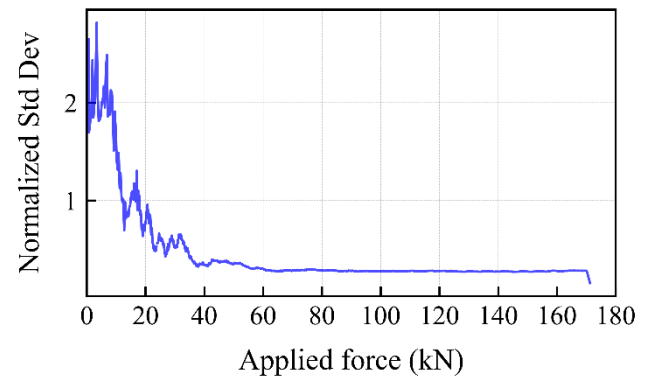


Figure 4-9: Deformation in the Y-Y direction for DSJ31 of the row near the joint versus the applied force.



(a)



(b)

Figure 4-10: normalized strain (i.e. measured strain/mean strain) of 9 horizontal data points in the middle of the joint: (a) Normalized strain versus applied force, (b) Standard deviation of the normalized strain versus the applied force.

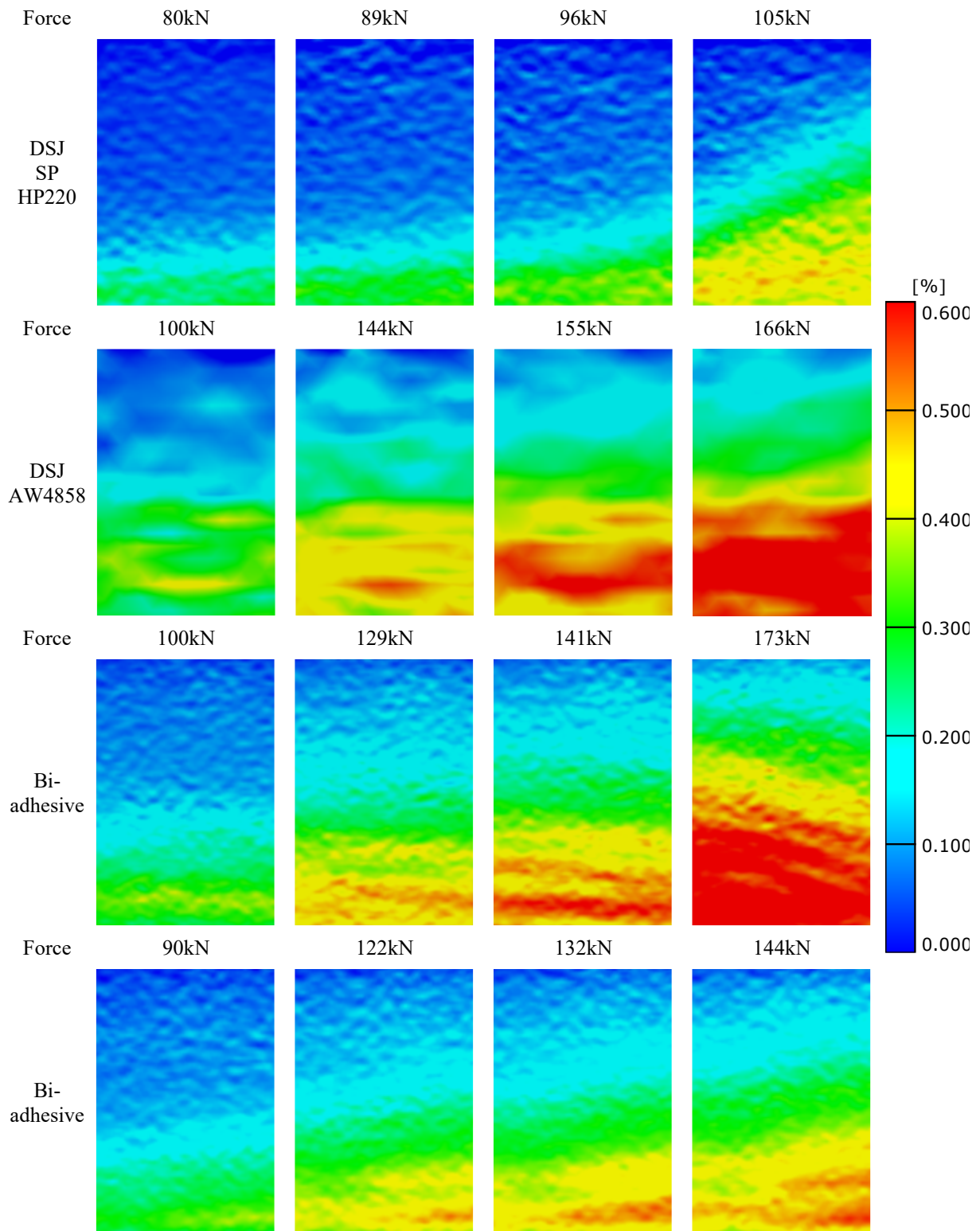
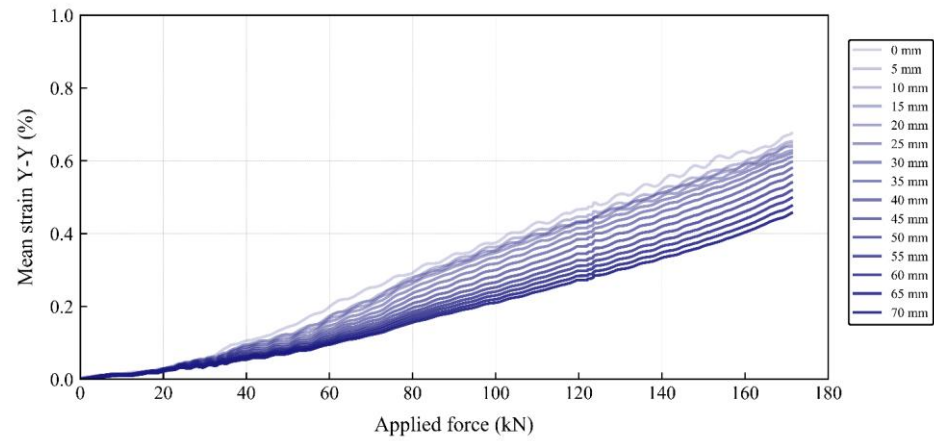
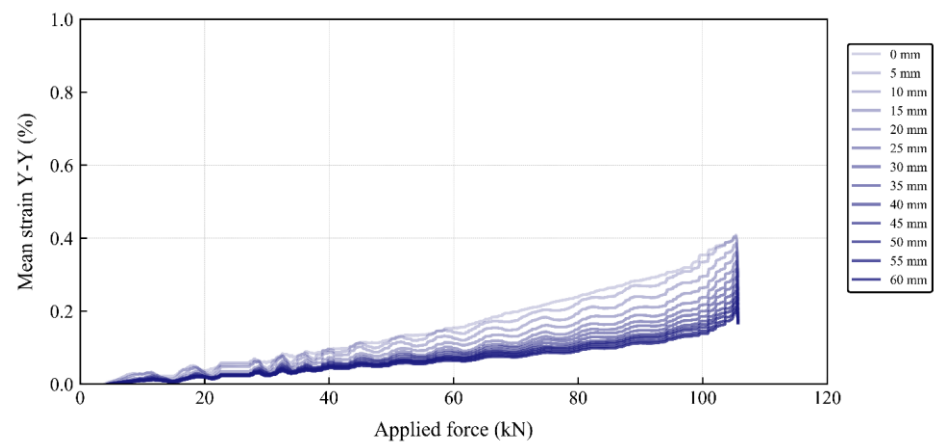


Figure 4-11: Strain fields on the CFRP back face for the double strap joints with the different adhesives.

a) DSJ with
AW4858/Hw4858.



b) DS
J with SP
HP220



c) DSJ with
bi-adhesive.

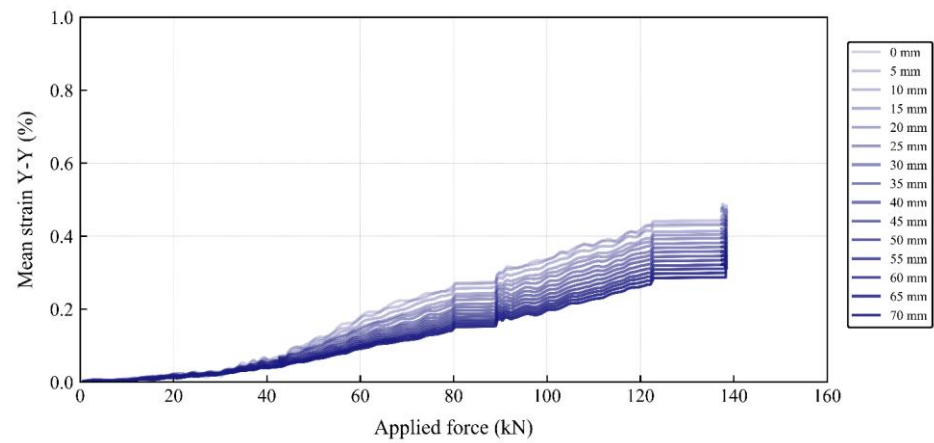


Figure 4-12: Mean smoothed strain measurements on the CFRP back face.

4.6 Failure pattern assessment using DIC

Figure 4-13 presents the evolution of the equivalent strain in the adhesive layer and the steel plate obtained using DIC. The strain fields in the thickness of the CFRP could not be captured due to significant noise. It should be noted that it is very challenging to analyse and obtain precise results in the adhesive layer [6]. Inhere the strain fields are provided for qualitative analysis only. Referring to Figure 4-13, the strain fields increase in intensity and cover a larger area as the loading increases. The failure of the specimen initiated in the most stressed area at the middle of the joint. The behavior of the joint shows unsymmetric behavior, where the lower adhesive layer is subjected to higher deformation level. This can be also observed in a similar test conducted by Haghani [3]. It might be due manufacturing imperfections or defects such as voids in the adhesive layer as highlighted in the figure. Voids are an inherent nature of bonded joints, it is impossible to manufacture a bonded joint free from voids [1]. As whether these defects are critical depends on their extent, position, and the nature of the applied stress. Wang et al [21] tested epoxy bonded lap joints with inserted large defects and showed that the joint strength little changed. The evolution of the equivalent strain of aligned points in the adhesive layer spaced with 5mm versus the applied loading is presented in Figure 4-14. The level of deformation is higher for data points closer to the joint, except at the end of the overlap, where in the last two points at 69mm and 64mm from the joint, the equivalent strain is higher due to peeling deformations.

From Figure 4-11 and Figure 4-13, the failure initiates and propagates from the middle of the joint until reaching the complete failure. This gives indication that that the peeling stress at the end of the CFRP does not have a contribution to the crack initiation. In the middle of the joint, the adhesive layer is mainly subjected to shear and compression stressed. This behavior is different from the ubiquitous single lap joints, or the double strap joints [12] with very short overlap used in the aerospace structure crack repairs, where the effect of peeling stress can be significant. So, this confirms the previous statement that the behavior of the bi-adhesive joints used in this study and the strengthening strategy is fundamentally different. Inhere, the ductile adhesive in the middle of the bi-adhesive joint is the main

contributor to the joint strengthening by allowing a more uniform distribution of stresses. The results presented in this study show that the use of bi-adhesive in the shear region given a high strength tough ductile adhesive is used can provide a significant improvement to the performance of the joint.

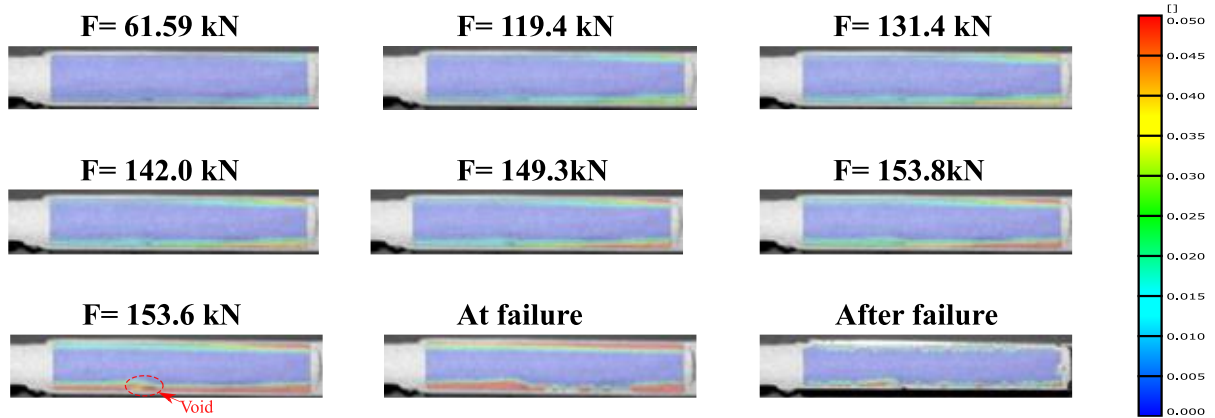


Figure 4-13: Equivalent strain distribution in the adhesive layer of specimen with AW4858/HW4858 adhesive using DIC.

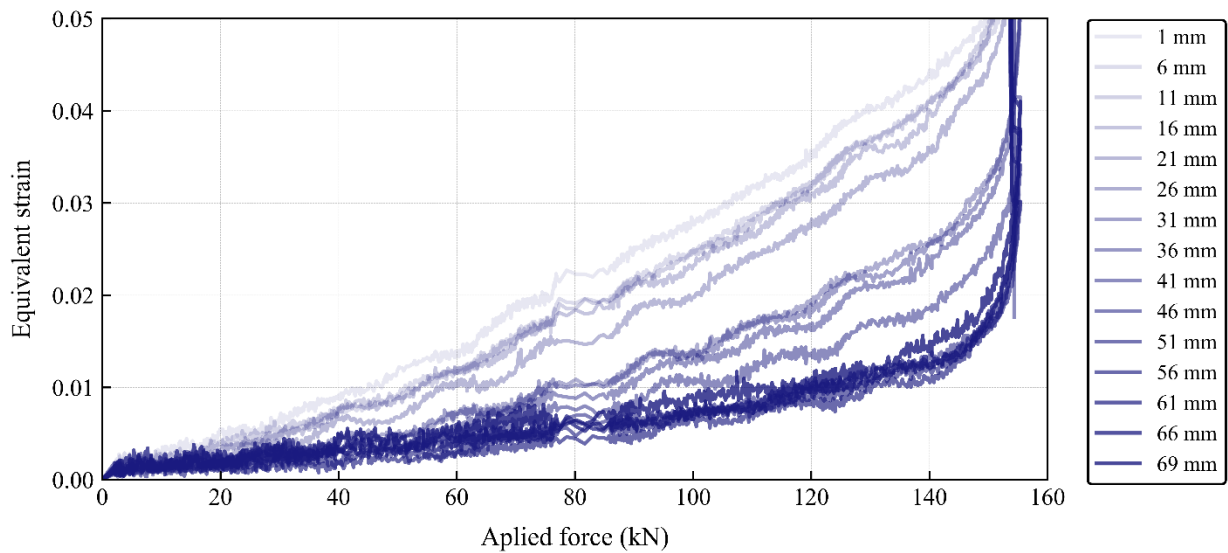


Figure 4-14: Equivalent strain at different points from middle of the joint versus applied force.

4.7 Joint strength efficiency

Figure 4-15 (a) compares the joint strength of the specimens made with the different adhesives. Specimens with brittle adhesives (i.e. SK30 and SP) reached an average strength of 76kN and 113kN with a standard deviation of $\sigma = 6.02kN$ and $9.25kN$, respectively. The specimens with the tough adhesive AW4858/HW4858 reached higher average strength of 160kN and $\sigma = 6.09kN$. Bi-adhesive joints achieved an average strength of 154kN with $\sigma = 12kN$. Compared to specimens with brittle

adhesive (S&P), an average of 36% improvement could be achieved when using bi-adhesive joints. The minimum design yield and ultimate stress of S275 steel are 275MPa and 370MPa, respectively. For the steel bars of 50x10mm used herein, the yield and ultimate strength become 137kN and 185kN, respectively. As shown in Figure 4-15 (a), the yield strength of the steel was exceeded for specimens with tough and bi-adhesive. The joint efficiency often used for welded joints refers to the strength ratio of the joint with respect to the strength of the base metal. A Joint Efficiency of 100% indicates that the joint has the same ultimate strength of the base metal. Figure 4-15 (b) shows that the specimens with SK30, S&P, AW, and bi-adhesive achieved an average joint efficiency of 41%, 61.4%, 86.53%, and 83.74%, respectively. Clearly, the tough adhesive and bi-adhesive joints have higher joint efficiency that is comparable to welded joints. The joints efficiency of the DSJ joint may be further improved by using tougher CFRP matrix to increase the interlaminar strength and thicker CFRP laminate to redistribute the stress. Thin CFRP plates (i.e. 1.4mm) has been used in this study and overall the specimens with AW and bi-adhesive achieved very high performance with a strength beyond the yield strength of the steel bars.

If the joint strength exceeds the yield strength of the parent material, the design criteria can be significantly simplified using joint efficiency ratio or similar parameter based on the parent material properties, without the need to carry out cumbersome design checks on the fracture of the adhesive layer or the CFRP, which is very complex. So further development of design rules based joint efficiency are higher recommend in future research.

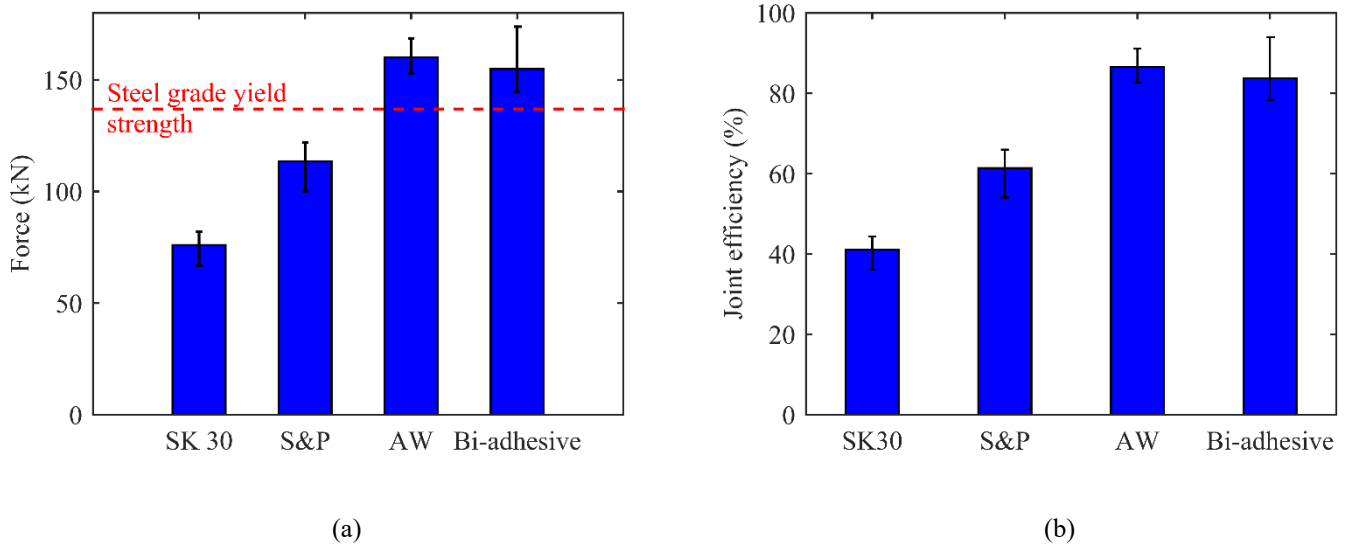


Figure 4-15: Strength and joint efficiency: (a) comparison of the joint strengths versus the nominal design yield strength (i.e. $\sigma_y = 275\text{MPa}$), (b) joints efficiency based on design ultimate strength (i.e. $\sigma_u = 370\text{MPa}$).

4.8 Numerical analysis

A 2D Finite element model was developed in the commercial software ABAQUS to simulate the behaviour of DSJ specimens. Only half of the joint was modelled due to symmetry. The model is shown in Figure 4-13, which consists of a steel bar, adhesive layers, CFRP plates, and testing machine grips. Since yielding was observed in the load displacement curves of some specimens with the tough adhesive, the machine gripes were also modelled in order to capture any stress concentration zone created by the compression of the grips. The numerical model has the same geometry and dimensions of the experimental specimens shown in Figure 4-3. All the model was developed using plane stress linear quadrilateral elements CPS4R available in the software library. The elasto-plastic material behavior of the steel and the adhesives was modelled using isotropic model calibrated using experimental true stress-strain curves. The CFRP was assumed linear elastic with mechanical properties provided by the manufacturer as presented previously. The CFRP has a strong unidirectional behavior following the direction of the fibres, and a weaker behavior in the other directions, which is mainly governed by the behavior of the matrix. Hence, it is more appropriate to model the behavior of the CFRP using an orthotropic material model. However, due to the unavailability of experimental data, the CFRP plates were assumed isotropic. This assumption is supported by the study of Haghani [22]

who investigated the effect of considering orthotropic or isotropic material models in ABAQUS. His results showed that the isotropic material model gives acceptable results. A refined meshing was adopted in the numerical model with mesh size of 0.5mm for steel bar and 0.2mm for CFRP and adhesive layer, as shown in Figure 4-17. A surface to surface contacted was assumed between the grips and the steel bar. A penalty friction isotropic contact was assumed in the tangential direction with a coefficient of friction of 0.6, which was the minimum to ensure no sliding as in the experiment. In the normal direction, a hard contact was assumed. A compression force of 1MN was uniformly applied on the grips as in the experimental data. The steel material of the grips was assumed linear with 200GPa . The free edge of the grips was constrained as a rigid body to a reference point “RP1” which was in turn restrained in all degrees of freedom to ensure fixed boundary conditions. The loading was applied on the CFRP edge section as only half of the joint is modelled. The CFRP edge section was constrained as a rigid body to a reference point RP2 where a displacement loading was imposed. The steel/adhesive interface was modelled with tie constraints where the steel surface is taken as master surface and the adhesive surface as slave. Similarly, for the CFRP/adhesive interface, the CFRP surface is taken as master and the adhesive surface as slave. Figure 4-18 shows that the numerical results match fairly well the experimental data, which validate the experimental results shown previously. Note the failure of the joint or any damage in the joint components is beyond the scope of this study. The numerical model presented in this section is mainly to check the distribution of stresses. Figure 4-19 shows the principal stress in the steel bar at the maximum applied force of 152kN , the maximum stress is almost 340MPa which represents 97% of the experimental yield stress. This indicates that at this stage the steel did not experience any plastic deformations but at higher loads, the steel would yield, which is the case of the specimen DSJ-AW that experienced significant yielding as shown in Figure 4-6 (c). Figure 4-20 shows the normal and the shear stress in the CFRP/adhesive interface where failure occurred in the experiments. As can be seen, the joints with the brittle adhesive SP HP220 exhibit higher normal stress than the joint with tough adhesive AW4858/HW4858. Similarly, in the shear behavior the joint with brittle adhesive shows higher level stress near the joint region whereas the joint with tough adhesive AW4858/HW4858 experience a more distributed stress and much lower in intensity. Clearly, the effect

of bi-adhesive can be seen in Figure 4-20 where the use of this technique reduces significantly the level of stress and promote more uniform distribution.

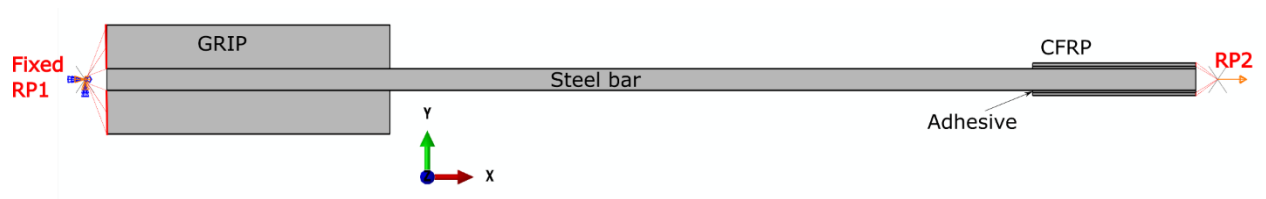


Figure 4-16: Geometry of the numerical model.

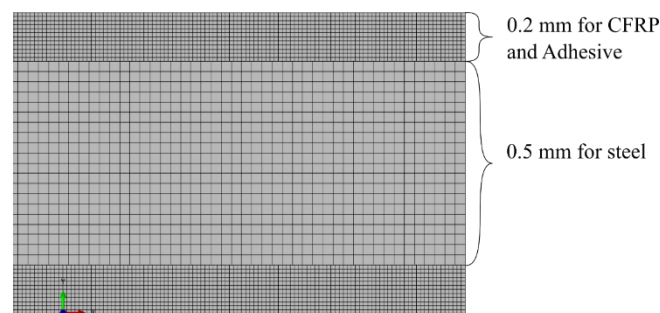
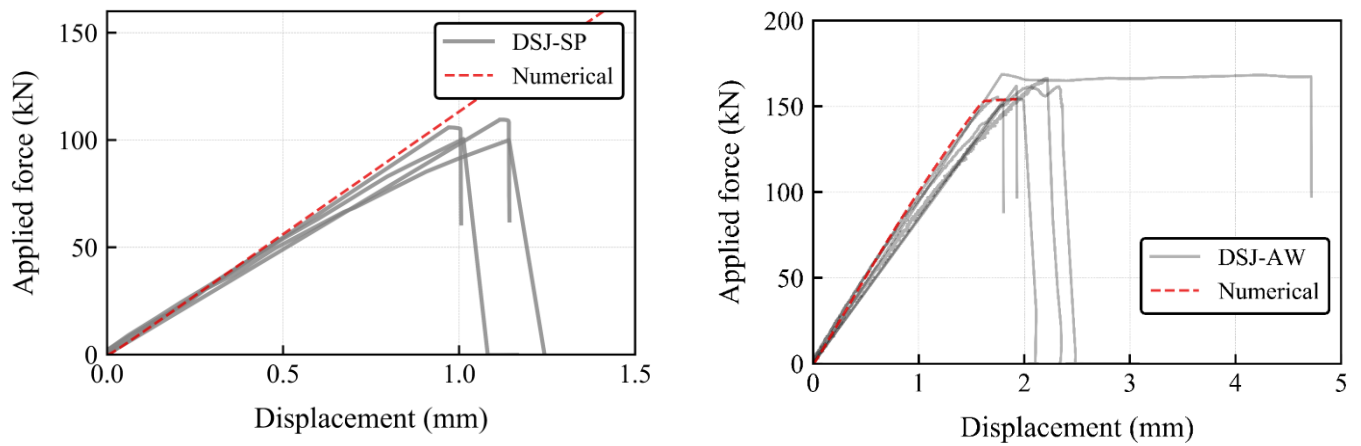


Figure 4-17: Mesh size.



(a) DSJ with SP HP220

(b) DSJ with AW4858/HW4858

Figure 4-18: Comparison between numerical and experimental results.

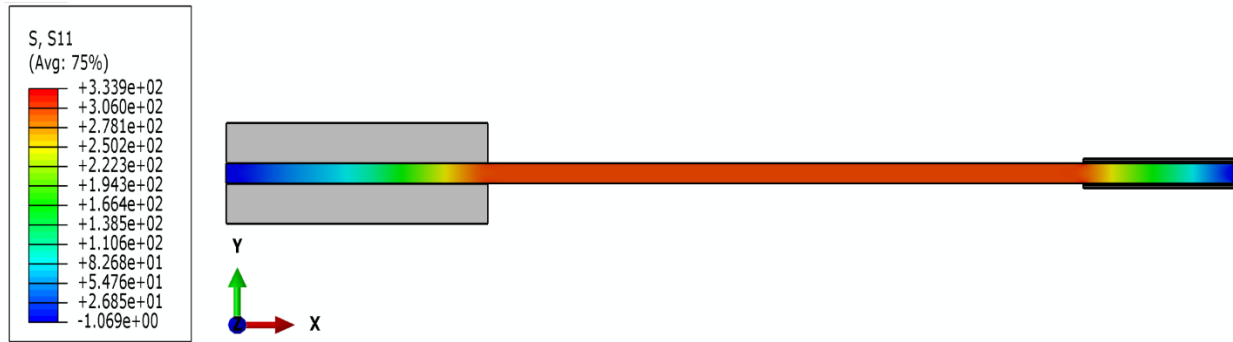


Figure 4-19: Principal stress in the Steel bars at 152kN.

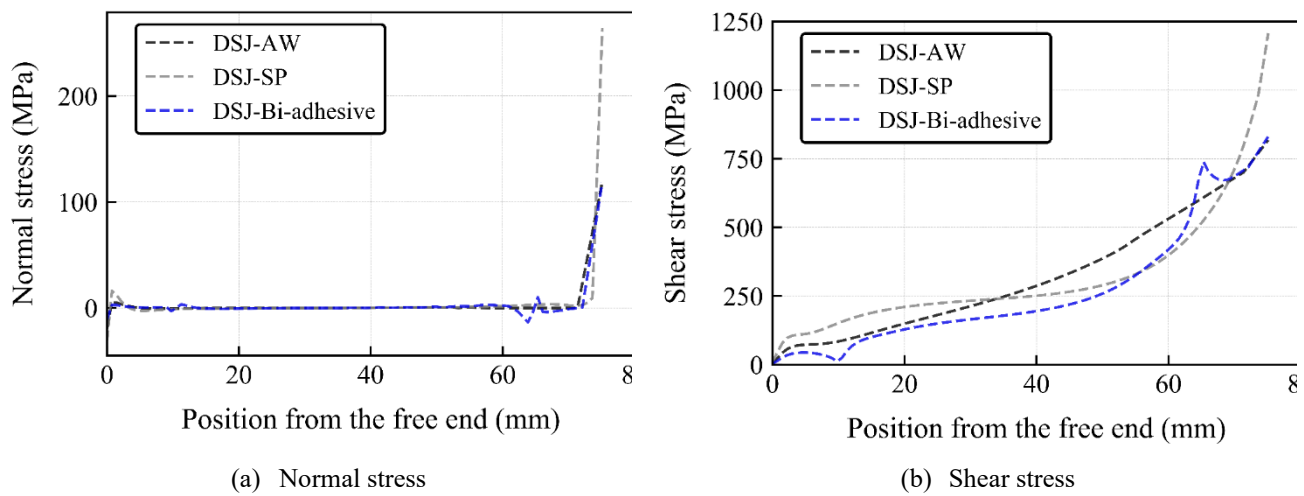


Figure 4-20: Stress state at the CFRP/adhesive interface.

4.9 Conclusion

This chapter presents an experimental and numerical study on the behavior of adhesively bonded CFRP/steel joint for the repair of metallic infrastructures. A comparison between brittle adhesives commonly used in related research and an extremely tough adhesive that combined high strength and high ductility is provided. The results showed that:

- tough adhesives provide much higher performance than the brittle adhesives and are more suitable for bonding on metallic components.
- However, the cost of the tough adhesives can be significantly higher than brittle ones. Therefore, a cost effective bi-adhesive joint is proposed for the repair of metallic infrastructures.
- The proposed bi-adhesive joint promoted a more uniform stress distribution on the back face of the CFRP and reached higher strength than the joints with brittle adhesive.

- The performance of the bi-adhesive joint was almost similar to the joint with the tough adhesive.
- Some specimens exceeded the minimum yield strength of the steel bar used in design codes. Therefore, the use joint efficiency ratio related to the minimum yield or ultimate strength of the parent material is highly recommended in future research. The use of the joint efficiency can simplify the design procedure significantly.

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PART II: The application of CFRP to extend the fatigue life of metallic structural details

CHAPTER 5: Fatigue strengthening of old metallic bridges using CFRP: a literature review

5 Fatigue strengthening of old metallic bridges using CFRP: a literature review

Abstract

This chapter presents a literature review on the fatigue strengthening of old metallic bridges using CFRP. The state of the existing old metallic bridges is presented. case studies of CFRP repair interventions on old metallic bridges is presented. Extensive review and discussion on the studies conducted on fatigue strengthening of structural components using CFRP, including bridge girders and small-scale specimens representative to connection details. The influence of different parameters on the effectiveness of the CFRP strengthening is also discussion which includes: CFRP geometrical configuration, CFRP stiffness, the initial crack length, and mixed mode effect. The need of more studies related to the delay of crack initiation is highlighted.

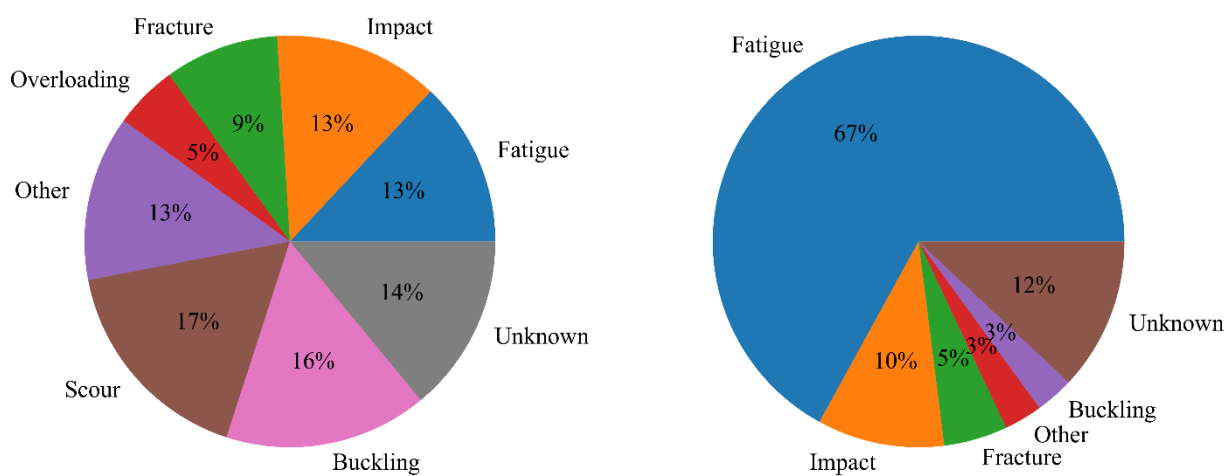
5.1 State of existing of old metallic bridges

The maintenance of aging metallic bridges around the world is a critical issue. U.S Governmental agencies classified 161,892 bridges of 600,905 as deficient. A similar situation is found in Australia and Japan. In Europe, 30% of the railway bridge stock has over 100 years old [1]. A large portion of the bridge stock was constructed during the late 19th and the beginning of the 20th century using riveting technology with old mild steel or puddle iron. Some exceptional landmarks made of cast iron can be found in the U.K.

Many old bridges exceeded the intended design service life. The nature and the level of loadings have changed significantly compared to what they were designed for. Old bridges suffer from but not limited to corrosion, fatigue cracks, buckling, and loss of connecting elements. A survey on metallic bridge failure that included 164 metallic bridges, reported by Imam and Chryssanthopoulos [2], showed that 87 cases completely collapsed, and 73 did not collapse but caused complete loss of function of the bridge. It was found that the fatigue failure represents only 13% for collapsed bridges, but covers the wide majority for non-collapsed bridges with a percentage of 67%, as shown in Figure 5-1. So in total from this data, at least 60 metallic bridges (36% of the total) have been damaged by fatigue loading,

which makes fatigue problem as the most critical failure mode for metallic bridges, excluding natural hazards, ship and vehicle impacts that are not selected in the data pool presented herein. Corrosion also represent a serious problem for aging metallic bridges however, it is not included in this survey.

Fatigue is one of the major concerns that requires lot of attention. Fortunately, the fatigue damage in ductile metals do not cause abrupt failure but rather show a slow increase of cracks' length until reaching a critical level. However, monitoring of the crack and strengthening intervention are very important to avoid a sudden failure.



(a) Failure modes of 87 collapsed bridges.

(b) Failure modes of 73 non collapsed bridges.

Figure 5-1: Metallic bridges failure statistics of 164 cases (Imam and Chryssanthopoulos [2]).

5.1.1 Metals used for bridges

The materials and assembly technics used to erect metallic bridges have evolved significantly over time. The metals (i.e. cast iron, puddle iron, and mild steel) and riveting methods used a century ago, are not practiced for infrastructures nowadays. Consequently, the fatigue performance of old riveted bridges has not to been investigated to some extent as for current structures assembled by welding ref. [3], [4].

Figure 5-2 illustrates the evolution of the materials and assembly methods used for metallic bridges in Europe.

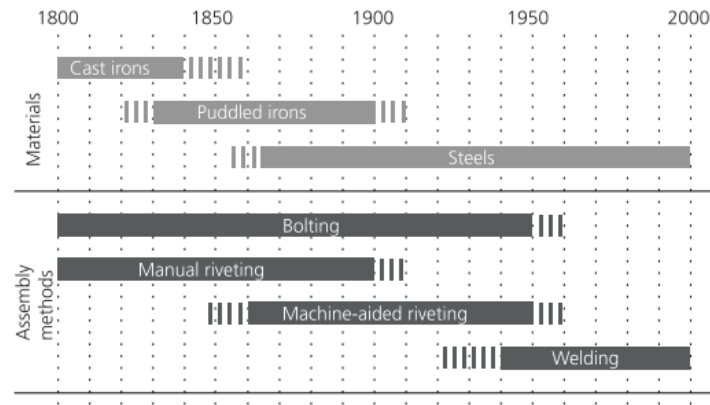


Figure 5-2: Evolution of materials and assembly methods used in metal bridges in Europe [5].

The material behavior of cast iron, puddle iron, and steel differ significantly [6]. Cast iron is brittle and weak in tension due to the coarse granular structure and the presence of inclusions. It was found surprisingly corrosion resistant but drilling is too risky and welding is impossible. This makes retrofitting of cast-iron structures quite challenging since it is almost impossible to use conventional repair methods such as bolted or welded cover plates. Fortunately, only a few bridges made of cast iron still exist nowadays [7]. On the other hand, puddle iron is a ductile material but with a behavior that is highly directional depending on the orientation of slags fibres. Lesiuk et al [8], tested puddle iron specimens under mixed-mode fatigue crack growth and showed that the influence of inclusions on the crack initiation angle is significant. Another well-known drawback of the puddle iron is that it suffers from serious problems of corrosion. The combination of current steel and puddle iron makes things worse because puddle iron has slightly different electro-potentials that lead to galvanic corrosion [6]. Old mild steel used in the early 20th century is similar to current carbon steel S235 [3]. Zerbo et al [9] resumed the shared typology of damage observed in bridges made of each material:

- Cast iron: the common damages are the appearance of the crack due to the impact of heavy bodies and to thermal starts that induce tension in the member.
- Puddle iron: a reduction of section tensile resistance due to corrosion, is usually observed.
- Mild steel structures of the 20th century: the most common causes of deterioration are due to the lack of suitable maintenance, corrosion, and damage in fatigue sensitive details.

A thorough description of the material behavior of puddle iron can be found in de Jesus et al. [10]–[14] and in Correia [15]. Extensive experimental works were conducted in the Faculty of Engineering University of Porto for materials extracted from five operating bridges in Portugal, where four of them are made of Puddle iron and one of mild steel. The bridges considered are namely: Eiffel Bridge (1878), Luiz 1 Bridge (1886), Fão Road Bridge (1892), Pinhão Road Bridge (1906), and Trezói Railway Bridge (1956). The main investigations on this old metals included: the mechanical tensile properties of the materials, the chemical composition, low cycle fatigue data, details on the microstructures, elastoplastic stress-strain behavior, and the fatigue crack propagation. Observations on the microstructures of the 5 materials showed they are mainly composed of ferrite matrix. The puddle iron material showed heterogeneous microstructure which is consistent with heterogeneous chemical composition. All materials have lined-up inclusions induced by deformations that limits their ductility.

5.1.2 Residual fatigue life of old metallic bridges

Earlier investigations in Europe (i.e. between 1990 and 1995) on the residual fatigue life of riveted metallic bridges were conducted in Swiss Federal Institute of Technology Lausanne by Brühwiler [16] Keller et al [17] and in Chalmers university in Sweden by Åkesson and Edlund [18]. Where all of them reported a very low-stress range present in a metallic element that is below the fatigue threshold which explains the reason why these structures are still in service.

- Brühwiler [16] assessed the remaining service life of 8 consecutive railway bridges of identical construction. The bridges were constructed in 1894 with ancient mild steel. Field measurements reported that under actual traffic loads the maximum stress range is 25 MPa, which is below the fatigue limit of 52MPa for riveted connections. The fatigue resistance curve corresponding to AASHTO D and Eurocode 3 part 9 details category 71 are assumed to be suitable for riveted connections as recommended by Brühwiler et al. [7], based on experimental testing of riveted bridge girders that are shown in Figure 5-3.

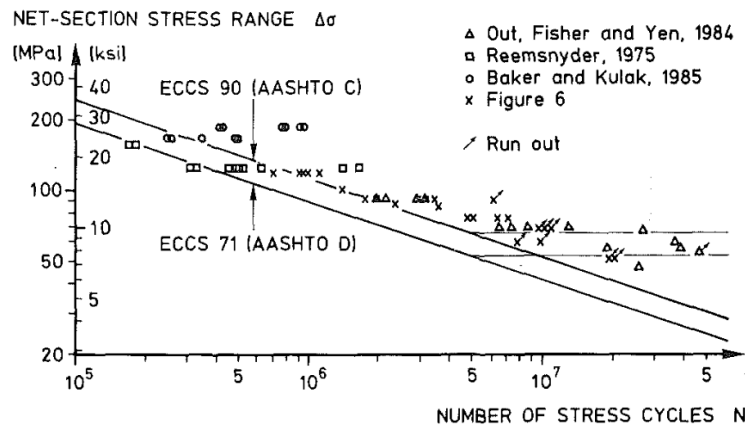


Figure 5-3: fatigue test results of full-scale riveted elements [7].

- Keller et al. [17] assessed the fatigue life of the oldest puddle iron bridge in continental Europe. The bridge crosses the river Rhine in northern Switzerland, built in 1859. Initially, the bridge was designed to carry two tracks but it only ever carried one track. Therefore, the predicted stress range was less than 31 MPa caused by the traffic of the suburban trains. The fatigue analysis was conducted assuming detail category 71 of Eurocode 3 part 9 as the fatigue resistance limit. The overall evaluation has shown that this bridge is still safe for future service mainly due the low applied loading.
- Åkesson and Edlund [18] tested the full-scale stringers and floor beams of a replaced riveted bridge, which was built in 1896 with old mild steel. The experimental program was conducted considering stress ranges of 40, 60, 80 and 100 MPa and stress ratio $R=0.14$ and 0.28 . The results confirmed that the detail category 71 of the Eurocode 3 part 9 fatigue resistance curve can be taken as a lower bound for the assessment of metallic riveted connections.

Later (between 1995 and 2010), more experimental studies were conducted on testing structural elements extracted from metallic riveted bridges. Most of them confirmed the adequacy of the fatigue resistance curve for detail category 71 in being a lower bound for metallic riveted connections.

- Helmerich (1997) [19] tested structural elements taken from three bridges in Berlin. The first one was built in 1902 on the Berlin underground railway U 1, it was made of old mild steel. The second one was a road bridge crossing the Teltow canal, built in 1906 with old mild steel. The last one was a suburban train bridge made of wrought iron built in 1890. The structural elements taken from

these bridges are two truss girders taken from the first bridge, three plate girders from the second one, and four plate girders from the last one. The fatigue loading was applied with a stress range that varied from 80 to 140 MPa. The results showed an agreement with the Eurocode detail category 71.

- Dibattista et al (1998) [20] presented all the previous tests on flexural members and axially loaded members and compared them with design codes provisions such EUROCODE 3 part 9 [21], AASHTO [22], and AREA [23]. The fatigue curve provided in AASHTO for category D, and EUROCODE for detail category 71, and AREA for category D for low clamping were found to be similar. All of them have the same slope of $m=3$ and a permissible stress range of 71 MPa at 2 million cycles. The only difference remains in the fatigue threshold due to the difference in the nature of the estimated applied loads. In AASHTO, the specifications are made for highways bridges, where occasional trucks greater in weight than the fatigue design truck are recognized. Thus, the permissible fatigue stress range is reduced from 48 to 24 MPa. Whereas AREA design code is made for railway bridges, the fatigue limit is set equal to 41 MPa. In Eurocode, the permissible fatigue range is 52MPa. The collected experimental results were all above the codes' fatigue resistance line, which confirms the adequacy of the assumptions made in the aforementioned studies.
- Al-Emrani and Åkesson (2002) [24] tested girders from a dismantled centenary bridge in Sweden under fatigue loading applied. The fatigue testing was applied in two steps. First, the girders were subjected to 10 or 20 million cycles at 60 MPa stress range until cracking was observed, then the loading was increased to 93-97MPa. The results showed a large scatter, but it was observed that the detail category C71 was suggested to be a lower bound estimation for fatigue life of stringers at the stress range 100MPa and $R=0.16$ and $R=0.2$.
- Taras and Greine (2010) [25] collected a large number of experimental data conducted on metallic riveted bridge elements that include small scale splices, full scale stringers, floor beams and struts in main girders. The experimental data are all plotted together in Figure 5-4. As it can be seen, the line 71 of EUROCODE 3 part 9 was found to be a good lower bound for the totality of the results.

The second line 100 (RL 805) shown in the plot is applicable to riveted members and connections made of steel fabricated after 1900 according to German RL805. As it can be observed, it lies above many tests results. The authors recognized that the line 71 of Eurocode is a good lower bound, but it is very conservative for lower stress range.

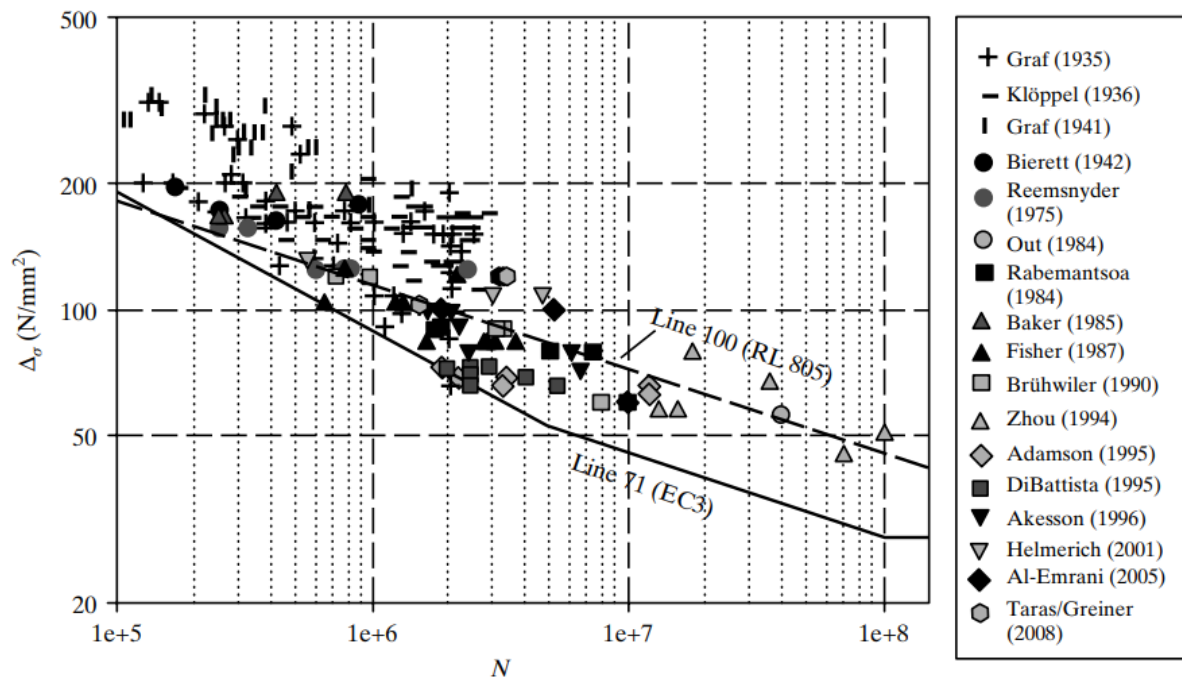


Figure 5-4: Collected test results on fatigue of riveted elements (Taras and Greine, [25]).

Most of the aforementioned experimental data were conducted on structural elements made of mild steel. Very limited data is for puddle iron elements. Brühwiler et al [7] recognized that the fracture toughness of puddle iron is lower than typical values for mild steel, and the critical crack lengths may be much smaller in puddle iron elements. Therefore, it was recommended whenever possible, an attempt should be made to obtain data corresponding to the material and possible crack orientation in the bridge element under consideration.

Recently (between 2010 and present), experimental campaigns (e.g. [26], [27]) were conducted on fatigue of shear splices made of existing puddle iron taken from bridges in Portugal and France. The results showed that the fatigue curve detail category 71 of EUROCODE 3 part 9, might be not adequate

to estimate fatigue life of riveted connections made of ancient puddle iron. Where very large scatter was observed for the same material under the same conditions.

Gallegos Mayorga et al. (2016) [26], [27] tested shear riveted connections extracted from a bridge over the Adour river (France), Fão Bridge (Portugal), Eiffel Bridge (Portugal). Specimens taken from the girders' web from Eiffel Bridge are shown in Figure 5-5. The totality of the experimental results is compared with the fatigue curve category 71 from Eurocode 3 part 9 in Figure 5-6. In conclusion, it was shown that:

- (a) The fatigue curve of the detail category 71 does not represent a conservative lower boundary for stress ranges over 150 MPa. Nevertheless, this stress range is quite higher than the level of the stress range present in real metallic bridges.
 - (b) For high-cycle fatigue (beyond the millions of cycles), the detail category 71 curve seems to significantly under-estimate fatigue resistance of the assemblies.
- i. Pedrosa et al (2019) [28] conducted fatigue tests on shear riveted connection made of puddle iron and mild steel and confirmed the inadequacy of the fatigue curve for category 71 provided by the EUROCODE 3. The specimens were made of material extracted from bridges in Portugal namely: Luiz I Bridge, Trezói Bridge, and Pinhão Bridge. The results revealed that some specimens showed lower performance than the Eurocode fatigue curve category 71 for low-stress range between 50 and 100MPa, as presented in Figure 5-7. The authors attributed this evidence to the presence of cracks originated by the prior operation of the bridge or material degradation.



(a)



(b)

Figure 5-5: Riveted specimens from the Eiffel Bridge (Portugal), original joint from the existing bridge (Gallegos Mayorga et al [26], [27]).

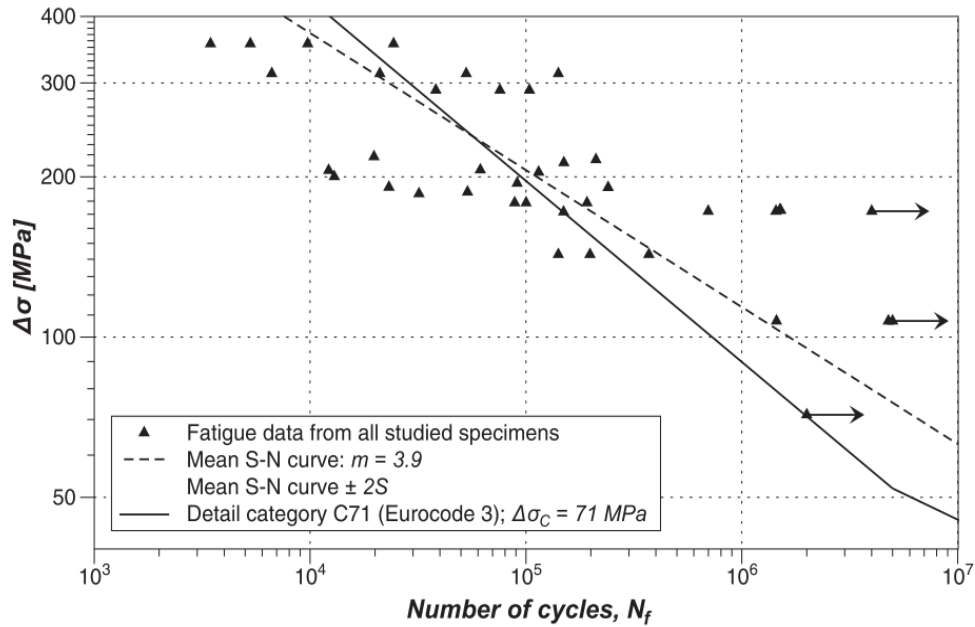


Figure 5-6: Fatigue tests compared with S-N curve for detail category 71 from EUROCODE 3 part 9; the dashed curve is obtained using statistical analysis (linearized boundaries method) (Gallegos Mayorga et al [27]).

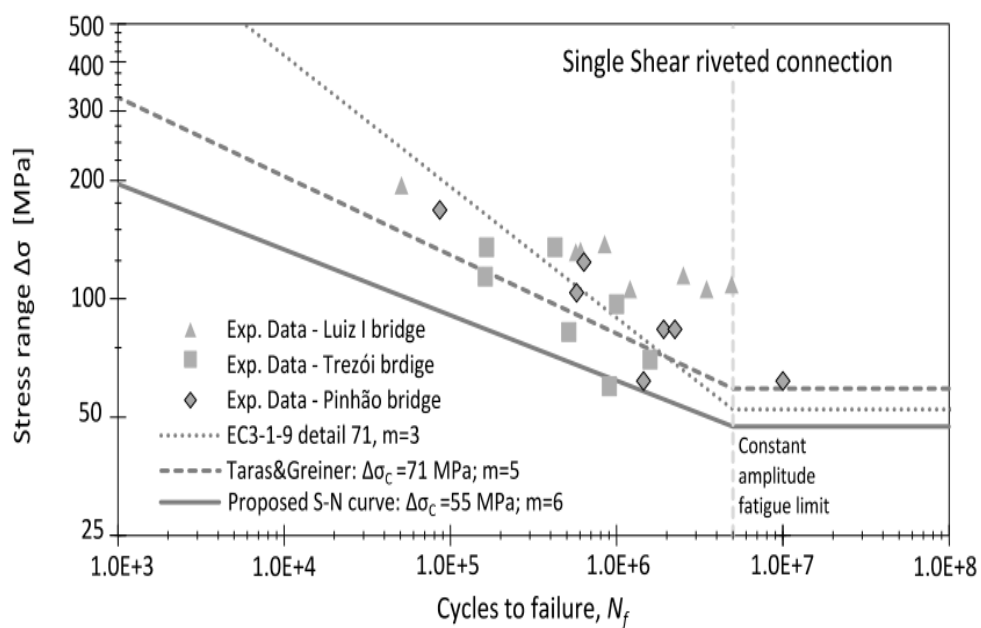


Figure 5-7: Proposed design S-N curves for single shear riveted connections from Old Portuguese metallic bridges (Pedrosa et al, [28]).

5.2 Case studies of metallic bridges strengthened using CFRP

This section presents some of the few case studies on the fatigue strengthening of metallic bridges using bonded CFRP implemented in practice. A review of most existing metallic structures reinforced with CFRP can be found in Harries 2011 [6] and in Hollaway (2008) [29]. However, inhere more recent repair interventions are included with focus on old metallic bridges only.

5.2.1 Diamond creek bridge, Australia [30]

Diamond creek bridge was built in 1896 was strengthened using bonded CFRP plates in the cross girders. Truck tests performed on the bridge demonstrated that the bending stress was reduced by 15%. The straim measurement on the back face of the CFRP revealed that the shear stress in the adhesive layer was approximately equal to 0.45MPa which is far less than the shear strength (i.e. 17.4MPa).

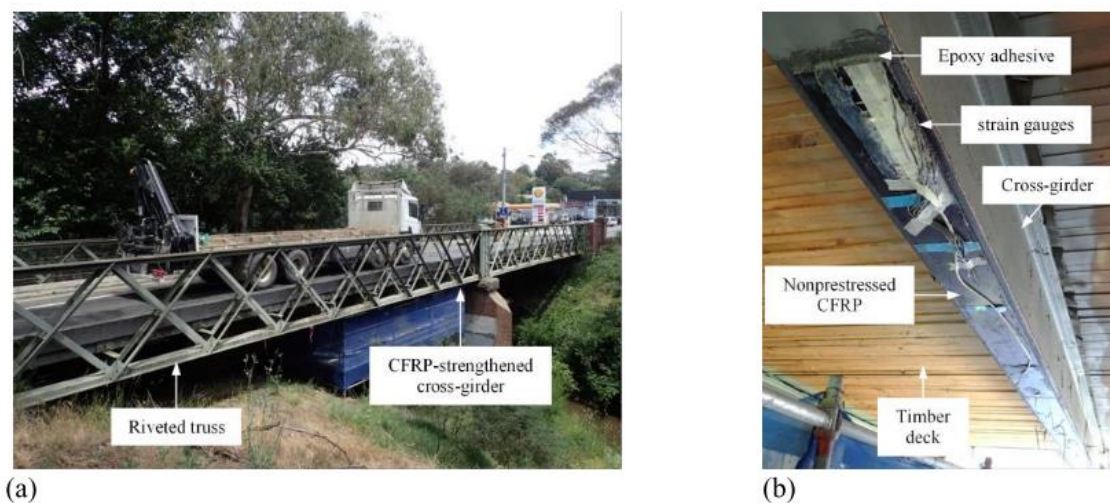


Figure 5-8: Strengthening of Dimond Creek Bridge in Australia using CFRP [30].

5.2.2 Jarama Bridge, Madrid [31]

Jarama Bridge, shown in Figure 5-9, is a steel bridge built in 1965 with hot rolled steel similar to S275 European steel. The CFRP strengthening was implemented in 2017 as a preventive fatigue strengthening since the resulting stress concentrations in the butt-welded splices could produce a

reduction in the fatigue capacity of the girders and 11 of the in-shop welds failed to meet the quality standards under AL2.



Figure 5-9: Jarama Bridge, Madrid.

5.2.3 Acton Bridge, London [32]

Acton Bridge, London is a riveted metallic bridge made with mild steel was strengthened for fatigue problems using bonded CFRP. The train wheel loads were directly applied to the cross girders which were susceptible for fatigue problem due to the frequency of the trains that runs every few minutes. Ultra high modulus CFRP was used to strengthen the cross girders. Figure 5-10 shows the repair intervention on the cross girders. Demonstration project indicated 24% reduction in live load stresses .



Figure 5-10: Fatigue strengthening of Acton Bridge using CFRP [32].

5.2.4 Slattocks Canal Bridge, Rochdale, UK [29]

Slattocks Canal Bridge, shown in Figure 5-11, is a historic steel bridge heavily trafficked was strengthened to increase its load capacity from 17 t vehicle to 40 t. After several proposal of strengthening techniques. the CFRP scheme was adopted since it preserves the historical aspect of the bridge. The CFRP repair intervention was operated without interruption of traffic.



Figure 5-11: Slattocks Canal Bridge CFRP repair intervention.

5.2.5 Hythe bridge, oxford UK [33]

Hythe bridge was in 1861 with two span simply supported cast iron girders and brick jack arches. It was strengthened in 1999 using prestressed bonded CFRP plates on the girders to increase the load capacity to 40t vehicles. The objective of the strengthening was to raise the capacity to 40 t vehicle. The bridge is located in a busy road in the city centre. The strengthening intervention did not interrupt the traffic.

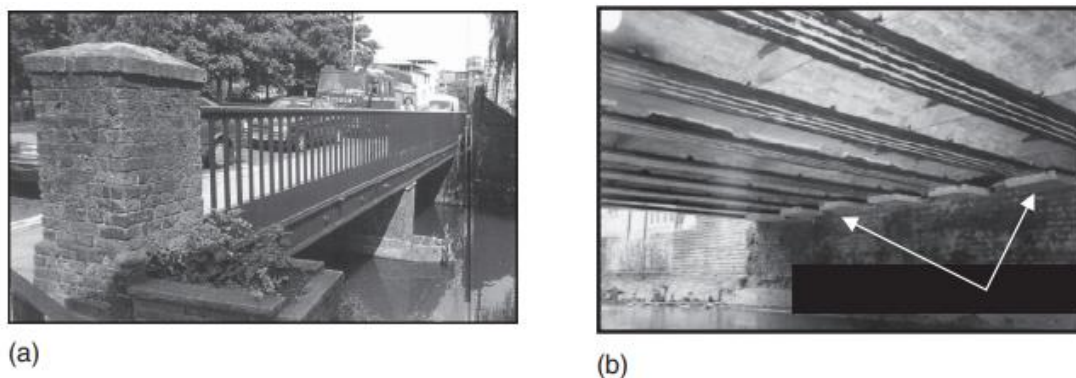


Figure 5-12: CFRP strengthened Hythe Bridge, Oxford, UK [33].

5.3 Fatigue strengthening of structural components using CFRP

5.3.1 Fatigue tests on cracked girder reinforced by CFRP

The first experimental work conducted on the application of CFRP to old metallic bridges to extend their fatigue life was done by Bassetti et al. [34] who took the opportunity to test six girders of a dismantled railway old bridge that was built over the Hinterrhein River, Thusis, Switzerland in 1901. The experimental program consisted of testing 4 girders under fatigue loading to obtain their residual fatigue life and strengthen three girders with CFRP laminates to verify the effectiveness of this repair technique. Before applying reinforcements to the girders, Bassetti et al. [34], [35] tested central hole small scale specimens reinforced with non-prestressed and prestressed CFRP laminate, in order to determine the best strategy to adopt. The results showed that prestressed CFRP provided the best fatigue improvement. Therefore, prestressed CFRP laminates were adopted as a retrofit scheme for upgrading the three girders.

Bare girders without reinforcement were subjected to fatigue loading with a stress range equal to 80MPa and a ratio $R=0.1$. The cracks occurred after being subjected to 3.5 million cycles at the edge of rivet holes at the zone of the maximum moment (i.e. mid-span of the girder). The measured cracks had a length between 4mm and 10mm from the rivet head, as shown in Figure 5-14, where the initial pre-crack is illustrated at zero number of cycles. Then, the strengthening was applied using prestressed CFRP laminates, the girders were subjected again to fatigue loading. The crack propagation of the three repaired girders and four non-repaired girders is shown in Figure 5-14. The results show that the retrofit scheme could completely stop the crack propagation, where the crack growth did not increase even after 20 Mio cycles.

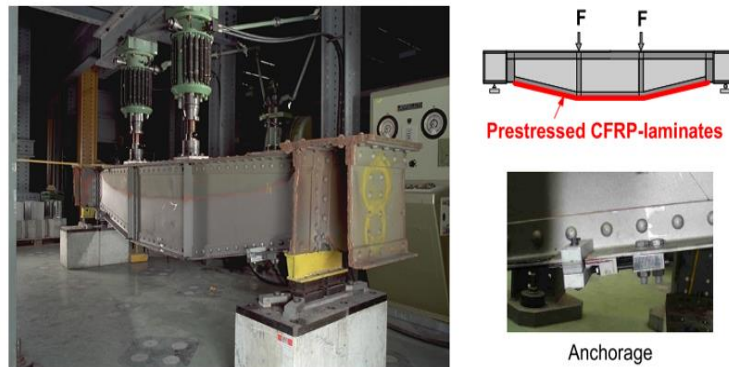


Figure 5-13: repair girder with pre-stressed CFRP laminates [35].

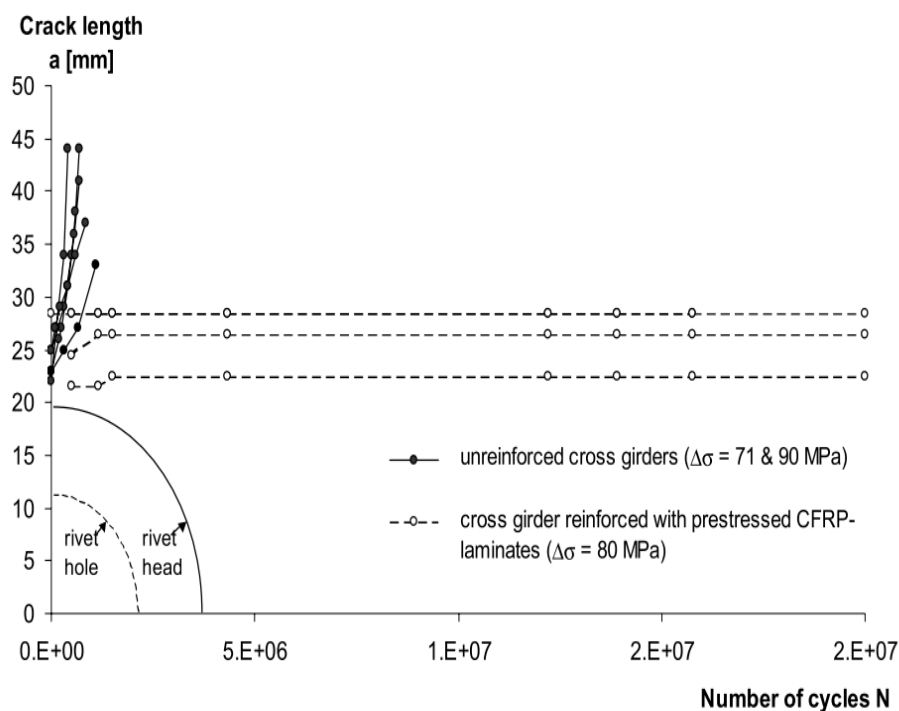


Figure 5-14: The crack length versus the number of cycles for repaired and non-repaired specimens.

Tavakkolizadeh and Saadatmanesh [36] tested 21 steel beams under fatigue loading to investigate the effectiveness of the CFRP retrofit scheme in extending the fatigue in aging steel flexural elements. Repaired and unrepaired specimens were tested using the four-point bending rig configuration with loading frequency between 5 and 10Hz. The specimens were made of A36 steel grade, featuring an artificial notch of 12.5mm length and 0.9mm width in the tension flange. The fatigue loading included

different stress ranges between 69 and 379 MPa. The results revealed that the application of CFRP extended the fatigue life by a factor ranging between 2.5-3.4.

5.3.2 Fatigue tests on cracked small scale reinforced specimens made of old metals extracted from existing riveted bridge

In this section, experimental works from the literature conducted on small scale specimens extracted from old riveted metallic bridges will be presented. The experimental campaigns conducted by each research group will be described in the following sub-sections separately.

5.3.2.1 Bassetti et al. ([34], [35], [37])

As stated in the previous section, one of the benchmarks works on the application of CFRP in extending the fatigue life of old bridges is the research conducted by Bassetti et al. [34], [35], [37] who took the opportunity to test girders of a dismantled railway old bridge built in Switzerland in 1901. First, the experiment consisted of testing 4 girders under fatigue loading without any repair intervention. The results showed that the cracks occurred exclusively at rivet holes. Therefore, prior proceeding to the repair intervention, small scale specimens (i.e. metallic plate with a central hole) were tested under fatigue loading. Figure 5-15 (a) illustrates the specimen's geometrical configuration. Artificial notches of 5mm length and 0.1mm width were introduced to the specimens. To determine the best repair option, different types of CFRP laminates and levels of pre-stress were applied. Figure 5-15 (b) shows the evolution of the fatigue crack length against the number of cycles. Note that the initial length a_0 shown in the plot represents the distance from the center of the hole to the crack tip. The specimens were tested under constant amplitude fatigue loading with stress range $\Delta\sigma = 80\text{MPa}$ and a stress ratio $R = 0.4$. The results showed that the un-reinforced specimen failed after 300, 000 load cycles. The application of non-pre-stressed CFRP (SIKA CarboDur S512, $E=150\text{MPa}$) resulted in an increase of fatigue life by a factor of three. On the other hand, pre-stressed CFRP was much more effective where the fatigue life could be extended by a factor of 20 in the case of CFRP laminates SIKA CarboDur M612 ($E=210\text{MPa}$) and a pre-stress load equal to 41.2KN. It can be noticed in the plot that the increase of the CFRP stiffness whether by increasing the plate thickness or the elastic modulus has a positive effect in extending the fatigue life.

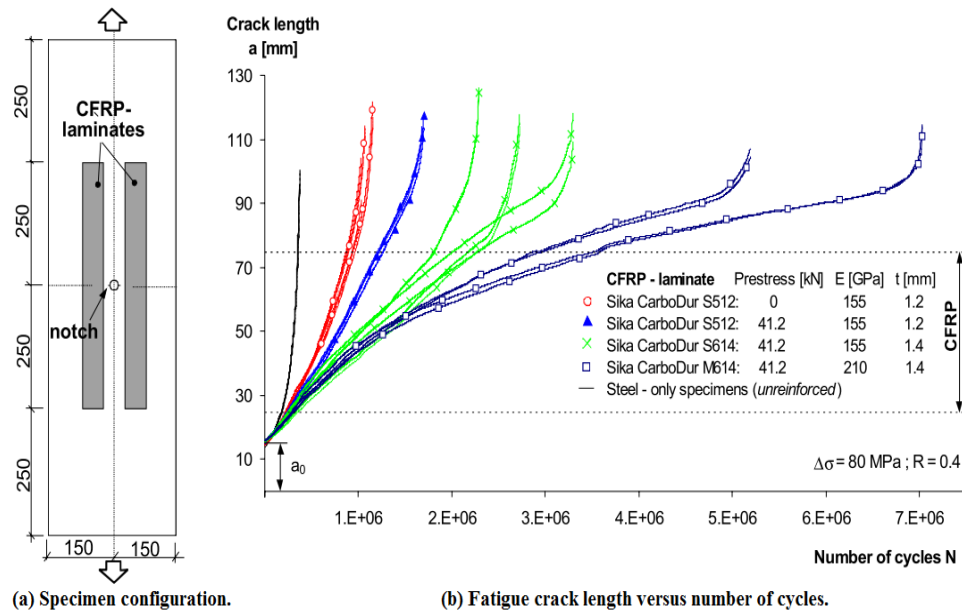


Figure 5-15: Fatigue testing of small scale specimens with different CFRP laminate types.

5.3.2.2 Täljsten et al. [38]:

Täljsten et al [38] also carried out an experimental study on notched central-holed metallic plates, quite similar to the one shown in Figure 5-15, with material extracted from old steel girder of a demolished truss bridge in Sweden, built in 1896. Non-pre-stressed and pre-stressed CFRP were bonded to the metallic specimens using two different adhesives (i.e. BPE 567 and Pox SK 41, with $G=2.4\text{GPa}$ and 3.8GPa , respectively) and two types of CFRP plates (E 50 C and M50 C, with $E=155\text{GPa}$ and 260GPa , respectively). The fatigue loading was applied with loading frequency equal to 14Hz , stress range equal to 97.5MPa , and maximum applied loading $F_{max}=175\text{KN}$. The results showed that the non-pre-stressed CFRP improved the fatigue life by a factor equal to 2.54-3.74 and pre-stressed CFRP stopped completely the crack growth. The stiffness of the laminates was also found to positively influence the fatigue crack growth and fatigue life.

5.3.2.3 Lesiuk et al. [39]:

Lesiuk et al [39] conducted experimental testing and numerical analysis to investigate the fatigue crack growth of CFRP strengthened CT specimens of old mild steel extracted from a railway riveted metallic bridge constructed in 1899-1902, in Kluczbork, Poland. The CFRP plates were applied using two configurations as shown in Figure 5-16. The CFRP plates are made of Sika CarboDur S1014/180 ($E>165\text{GPa}$), bonded with epoxy resin. The fatigue loading was applied by a sinusoidal loading with a

frequency $f=12.5\text{Hz}$ and a stress ratio $R=0.1$. The maximum applied force for the un-reinforced specimen was equal to $F_{max}=4.5\text{KN}$ and reinforced specimens were equal to $F_{max}=6\text{KN}$. The results of the experimental program showed an improvement of the crack propagation behavior though the delay of the crack length at failure “ af ”. In fact, the crack propagation of the un-reinforced specimen showed a failure between 200,000 and 250,000 cycles. Whereas, reinforced specimens experienced a stable constant crack propagation. The best performance was achieved by specimen “Reinforced Kluczbork 1” which had a full face covered configuration.

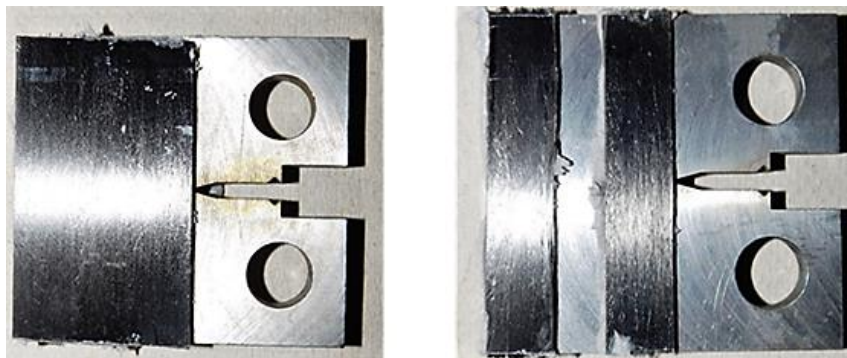


Figure 5-16: CT specimen strengthened with CFRP (Lesiuk et al [39]).

5.3.2.4 **Lepretre et al. [40]:**

Lepretre et al. [40] conducted a series of experimental tests on central notched plates made of puddle iron extracted from a dismantled Railway bridge (Pont de L’Adour) constructed in 1862 and on S235 steel assuming it is equivalent to old mild steel, as demonstrated by Cremona et al [3]. The specimens were similar to the ones presented in Figure 5-15 (a) but were slotted on one side only, assuming that applying two symmetric notches in the central hole as applied in previous studies is not well representative to cracked rivet holes where more often a single crack is present near the rivet hole. The notches had a width of 0.6mm and lengths of 7 or 13 mm to represent different degrees of damage.

Specimens were tested under different configurations including single-sided and double-sided patched specimens, using CFRP with different elastic modulus. The fatigue loading was applied with a ratio $R=0.1$ and a stress range of 90MPa. The results showed that the double sided configuration and higher CFRP elastic modulus provided the best fatigue life improvement.

5.3.3 Fatigue of cracked small scale specimens with current steel reinforced by CFRP

Recently, a very large number of research papers have been published to study the fatigue behavior of cracked small scale steel specimens reinforced with CFRP. Most of the studies replicated the pattern of the work made by Bassetti et al. [34], [35], [37] on small scale specimens, which consists of testing cracked small scale specimens and monitor the fatigue crack propagation. Then, compare the fatigue life of the bare steel specimens against the reinforced ones. Fatigue crack propagation curves of the studies presented in the following sub-sections will be not presented since they show very similar trends as the ones presented in Figure 5-15. Instead, the ratio of increase of fatigue life will be provided as the criterion of comparison. The novelty of the technical papers reviewed in this chapter remains in considering the effect of different influencing aspects such as the CFRP Configuration, CFRP stiffness, level of initial crack length, and the mixed crack propagation mode effect. Pre-stressed laminates are beyond the scope of this study. Within the framework of this thesis, an extensive literature review was conducted by consulting most of the technical literature related to the application of CFRP laminates in extending the fatigue life of small scale components. In this section, the most relevant findings related to the influencing parameters will be presented.

5.3.3.1 The effect of CFRP Configuration

Numerous research works [41]–[45] investigated the effect of CFRP geometrical configuration on the effectiveness in extending the fatigue life. The common experimental tests consist of testing central hole steel plates patch with CFRP, as shown in Figure 5-15 (a). The geometrical configurations consists of patching the CFRP on a single side or both sides of the metallic plate, varying the position of the CFRP with respect to the crack tip, and the CFRP width. The results are intuitive where double sided CFRP configuration with CFRP closer to the crack tip (or even covering the crack) and larger width provides the best performance. Single sided configuration provides low extension of fatigue life no more than a ratio of 2. In some situations it could even make things worse due to the bending induced by the extensibility of the CFRP.

5.3.3.2 Effect of CFRP stiffness

Alemdar et al [46] tested 15 central notched steel plates with varying CFRP overlays thickness to evaluate the effect of the ratio of axial stiffness on the fatigue life. The results showed that by increasing

the axial stiffness ratio from 0 to 0.4 compared to the parent material, the internal stress was reduced significantly and the fatigue life could be increased. This is inline with the results of several studies [42], [47], [48] which showed using multiple CFRP overlays or higher modulus CFRP increase the efficiency in extending the fatigue life.

5.3.3.3 Effect of initial crack length

Some studies [47]–[50] investigated the effect of CFRP application on specimens with different initial crack length to study feasibility of applying the CFRP repair for steel elements that already reached a significant fatigue damage state (i.e. long crack). For instance, Colombi et al.[47], [48] tested single notched edge steel plates with different initial crack length subjected to fatigue loading. All the specimens were initially slotted with a notch of 5mm long and 0.2mm thick. To simulate different damage levels, the bare steel specimens were subjected to fatigue loading until reaching the value of 6mm and 15mm of crack length which were considered later as initial damage level. Later, the reinforcement was applied by using CFRP plates ($E=210\text{GPa}$) bonded with epoxy adhesive. The results showed that specimens with short cracks achieved higher increase ratio of fatigue life extension. Yu et al. [49], [50] also tested specimens with different initial crack length by testing specimens with an initial crack that represents 2%, 10%, 20%, 30%, and 40% of the specimens' width. The results showed that the fatigue life was extended regardless of the initial crack length. However, earlier repair intervention is recommended.

5.3.3.4 Mixed-mode fatigue crack growth effect

Recently, few studies [51]–[56] investigated the fatigue mixed-mode effect on steel specimens reinforced with CFRP. The tests consist of introducing inclined initial artificial cracks to the steel specimens instead of cracks perpendicular to the loading axis as usual for mode I. Aljabar et al. [51], [52] were tested steel plates with a central hole and inclined crack reinforced by CFRP. The authors tested specimens artificially cracked with six different angles (10° – 90°) with respect to the loading axis. The results showed that specimens that have a crack angle > 60 were governed by fracture mode I. The mixed-mode was observed in specimens with a crack angle between 30 – 60° . The positive effect of the inclination angle was observed in both repaired and un-repaired specimens. The smaller inclination the

higher fatigue life was observed. Tao et al. [55], [56] also conducted experimental testing and numerical analysis to study the effect of the different parameters such the number of CFRP layers, CFRP width, CFRP modulus, and the load angle. The experimental results showed that the specimens with load angle 0° , 15° and 30° experienced an increase of fatigue life of 418%, 3524, and 158%, respectively. The repaired specimens in mode I had a better performance than the specimens in mixed mode.

2.1 Fatigue strengthening of non-cracked metallic details using CFRP

Little is known regarding the effectiveness of the CFRP repair technique in preventing crack initiation in metallic specimens. Most of the studies in the literature focused mainly on the CFRP repair of cracked structural elements. Certainly, the repair of cracked structural details is an important issue. However, it is equally important to study the fatigue strengthening non-cracked metallic elements. Due to longer fatigue life of non-cracked specimens compared to cracked specimen, debonding of the CFRP patch due to fatigue becomes more relevant. Debonding will depend strongly on the level of stress transferred to the CFRP laminates.

To the best knowledge of the author, only two experimental works conducted on non-cracked specimens were found available in the literature, namely the experimental tests on steel beams reinforced with CFRP to prevent crack initiation conducted by Ghafoori et al. [57], and the tests conducted by Wang et al [58] on open hole steel specimens reinforced by CFRP to upgrade the fatigue performance of open-hole connections.

Ghafoori et al. [57] tested four steel beams under four bending set up. Two small holes of diameter equal to 3 mm were introduced in the specimens by drilling in the bottom flange to create a zone of stress concentration, as shown in Figure 5-17. Three beams were strengthened with different CFRP laminates, namely, normal modulus (NM) CFRP, high modulus (HM) CFRP, and ultra-high modulus (UHM) CFRP with young modulus equal to $E=159$ GPa, 220GPa and 440 GPa, respectively. The fatigue loading was applied with a cyclic elastic loading range of $P= [1.7-18]$ kN. Since the main goal was to prevent the crack initiation; thus as soon as a crack was detected the specimen was considered to have failed. The results are presented in Table 5-1, where a clear improvement of the fatigue life was

achieved after the application of CFRP. Infinite life regime was achieved by applying HM and UHM CFRP in specimens B2 and B3. The specimen B1 retrofitted with NM CFRP failed after 1, 705 K cycles which represent 3.8 times the fatigue life of the bare steel specimen. The authors presented a design method based on CLD methodology to determine the required level of CFRP young modulus to prevent crack initiation by shifting the detail from finite life regime to infinite life regime.

Table 5-1: Fatigue test results of un-cracked steel beams reinforced with CFRP.

Specimen	Retrofit scheme	CFRP type	Cycles to failure	Failure mode
B0	Reference	-	4.48×10^5	cracked
B1	Bonded	NM	$1,705 \times 10^6$	Cracked
B2	Bonded	HM	2×10^6	Runout
B3	Bonded	UHM	2×10^6	Runout

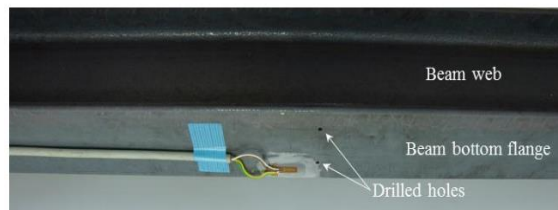


Figure 5-17: Drilled holes in the beam bottom flange (Ghafoori et al [57]).

Wang et al [58] tested non-cracked open hole steel specimens under fatigue loading. The specimens feature a hole occupying a the largest portion of the specimens width. CFRP plates with $E=250\text{GPa}$ were applied by covering the full face of the specimen except for the area of the hole. Some specimens were reinforced considering one CFRP layer, while others three layers. The fatigue loading was applied with a constant frequency of 8Hz, a stress ratio $R=0.1$, and a stress range between 200MPa and 340MPa. The full rupture of the specimen was considered as fatigue failure. All specimens experienced a fracture that propagated from the edge of the open hole to the external edge of the steel plate. Overall, the CFRP reinforcement provided some improvements with a lot of dispersion for higher stress levels. Specimens with three layers showed a better performance especially at a high level of stress range (i.e. around 300MPa).

5.4 Summary and discussion

This chapter presented a literature review on several aspects related to the fatigue assessment and retrofitting of old metallic bridges using CFRP. Where:

- i. A large number of experimental studies conducted on the fatigue assessment of old riveted metallic bridges were consulted and reported. It was shown that most of the tests were conducted on specimens with old mild steel and little were conducted on specimens made of puddle iron. The Eurocode fatigue resistance curve for detail category 71 was found to be a good lower bound for old mild steel riveted connections. However, this is not the case for puddle iron which showed fatigue data with a very large scatter and in some cases below the curve for detail category 71.
- ii. A survey on the application of CFRP to extend the fatigue life of metallic components is provided. Some examples of fatigue testing of full-scale specimens reinforced with CFRP were presented. An extensive state of the art review is provided on small scale specimens reinforced using bonded CFRP. A large number of experimental programs are available in the literature. The most relevant findings were summarized throughout this chapter by distinguishing between tests on old metals extracted from bridges and current steel, and also considering different influencing aspects such as the effect CFRP geometrical configuration, the effect of CFRP stiffness, the initial crack length, and mixed-mode effect.
- iii. Finally, a discussion is provided on the application of CFRP for non-cracked specimens to prevent or delay crack initiation. Very little is known on this topic. Due to the long fatigue life of un-cracked specimens, debonding problems due to the imposed fatigue loading may arise. Therefore, the fatigue resistance of bonded joint needs to be investigated too.

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Chapter 6: Statistical fatigue assessment of CFRP retrofitted steel structural details

6 Statistical fatigue assessment of CFRP retrofitted steel structural details

Abstract

The prevention of fatigue damage in aging metallic bridges is a worldwide concern. The application of CFRP on fatigue prone components can reduce the stress concentration level and significantly improve the fatigue life. Most studies available in the literature focused on the application of CFRP to repair elements with existing cracks. Very limited studies investigated the improvement of the fatigue life of non-cracked elements. Experimental data on non-cracked open-hole specimens retrofitted with CFRP available in the literature were collected and discussed in this study. The application of CFRP significantly extended the fatigue life of the specimens at stress range $\Delta\sigma < 220\text{MPa}$ where the fatigue data points converge towards a fatigue limit. This is very promising results for retrofitting bridges since most of the ancient metallic riveted assemblies are subjected to stress ranges below 150 MPa. The ratio of fatigue life extension decreases while the stress increases due to the weakening of the bonds at the steel-CFRP interface. Therefore, the fatigue data of specimens retrofitted with CFRP follow a nonlinear trend in log-log scale depending on the stress level. This chapter focuses on the derivation of probabilistic fatigue resistance curves (P-S-N curves) for non-cracked open-hole steel specimens that represent steel plates in tensile riveted or bolted connections. Since the probability distribution can be hardly defined based on physical evidence, several statistical distribution functions were explored. This includes the following distribution functions: the Normal; the two-parameter Weibull; the Gumbel; and, the three-parameter Weibull using the model developed by Castillo & Fernández-Canteli (CFC). The comparison between the different models shows that the CFC model was the most suitable to predict the fatigue life of CFRP retrofitted specimens since the P-S-N curves fit well different trends of the fatigue data including the transition between medium and high cycle range. Besides, a fatigue endurance limit could be predicted using the CFC model. Finally, P-S-N curves derived using the CFC model are proposed for CFRP retrofitted specimens. These can be used for design purposes as well as for reliability analysis.

6.1 Introduction

The maintenance of old metallic bridges is a major concern for governmental agencies. Many bridges in Europe have been operating for more than a century and are subjected to higher loads and speeds than those for which they were designed [1]. The fatigue damage is particularly important because it was originally not considered in the design of these structures and the old metals tend to further deteriorate under variable stresses [2]–[4]. Several bridges will need strengthening interventions to extend their service life. Therefore, the development of efficient reinforcement techniques to reduce the likelihood of fatigue damage is of paramount interest.

Bonded CFRP on metallic components to extend the fatigue life is regarded as a very promising solution within the research community [5]–[7]. In this context, the application of CFRP to metallic civil engineering structures subjected to fatigue loadings can be divided into two categories, namely: CFRP crack repair and CFRP retrofit. The CFRP crack repair consists of applying the CFRP near the crack to reduce the stress intensity factor and increase the fatigue crack propagation life [8]. While CFRP retrofit is applied as a preventive measure to upgrade the fatigue resistance of existing structures meeting non-crack assumptions. In the latter case, the CFRP can be applied at the global level such as bonded CFRP on the bottom flanges of the girders to reduce the overall stresses in the structure or at the local level such as bonded CFRP on critical connections to reduce the stress concentration factor. Some examples of implementations of CFRP retrofits in metallic bridges can be found in [9]–[13].

In practice, it is usually preferred to apply the CFRP retrofit than repairing existing cracks. Most of the application of CFRP in bridges are applied as preventive measures [9]–[14]. However, laborious experimental works were required to implement the CFRP in the aforementioned case studies. Since the fatigue design codes, such as Eurocode 3 part 1-9 [15], do not provide fatigue resistance curves for details reinforced with CFRP, and very limited studies investigated this topic. Therefore, there is a clear need for the development of fatigue design rules for structural details retrofitted with CFRP.

In the last two decades, a lot of research efforts have been dedicated to developing design rules for CFRP crack repairs [16]–[25]. But, little attention was given to fatigue retrofitting to prevent or delay the crack initiation [26]–[29]. Monfared et al, [30] tested notched specimens without initial pre-crack

under fatigue loading, where some of the specimens were retrofitted with CFRP. The result showed that the fatigue life could be extended by a factor of 1.7-2.06 depending on the CFRP configuration, the surface treatment, and the stress range. Pastor et al. [31] investigated the effect of bonding composite patches before the crack appears in aluminium plates. The rationale behind consists of deviating the stress flow from the critical zone and reduce stress concentration. Thus, the critical zone can be relieved from crack appearance. The results showed a significant extension of the lifetime for patched over unpatched specimens. Except for higher loads where the adhesive layer progressively fails which reduces the performance of the retrofitting scheme. Ghafoori et al. [26]–[28] provided an analytical procedure to design CFRP plates such that the structural component is shifted from a “finite-life” regime to the “infinite-life” regime using the constant life diagram approach. Recently, Wang et al. [32] tested steel open-hole specimens retrofitted with CFRP under fatigue loading. The authors compared the experimental data against the fatigue strength categories of the AASHTO specifications [33]. It was shown that the fatigue life could be significantly increased especially at lower stress levels.

Fatigue test results of metallic elements usually display significant scatter. The composite patch will provide additional variations and uncertainties due to the performance of the bonding at the steel-CFRP interface. Therefore, probabilistic analysis is necessary to derive reliable design procedures. This study focuses on the derivation of P-S-N curves assuming different statistical distribution functions. The experimental data of open-hole specimens retrofitted with CFRP was collected from the literature. Using these data, P-S-N curves are derived using the method prescribed by ISO12107 [34] that assume that the logarithm of the fatigue life follows on the normal distribution, models that assume the two-parameter Weibull distribution [35] and the Gumbel distribution [36], and the three parameters Weibull model developed by Castillo & Fernández-Canteli (CFC) [37]. Finally, a comparison between the P-S-N curves derived using the different models is presented, and the model that correlated the most with the experimental data is suggested.

6.2 Experimental data collection

Very limited experimental data on non-damaged steel specimens retrofitted with CFRP are available in the literature. To the knowledge of the authors only Monfared et al. [30], Ghafoori et al. [29] and Wang

et al. [32] published experimental data on steel specimens retrofitted with CFRP without initial crack subjected to fatigue loading. Monfared et al. [30] tested 11 double-edge notched specimens retrofitted with bonded CFRP using one-sided and double-sided configurations. The specimens were fatigue loaded until debonding or CFRP rupture occurred. Ghafoori et al. [29] tested five steel beams featuring two holes in the middle of the span under fatigue loading. Three of the specimens were bonded (without additional clamps) with normal modulus, high modulus, and ultra-high modulus CFRP, respectively. The results showed a significant increase in fatigue life especially in specimens with High modulus CFRP, where the tests were stopped after 2 million cycles without the appearance of crack initiation. The experimental results of Ghafoori et al. [29] and Monfared et al. [30] were very useful for qualitative analysis however they were limited in terms of the number of tests for each detail, therefore, they could not be further exploited in this study for the derivation of fatigue P-S-N curves. The Experimental data of Wang et al. [32] conducted on open hole specimens retrofitted with CFRP were found suitable to conduct the fatigue analysis due to the following reasons:

- The fatigue loading was applied under a stress ratio equal to 0.1 and with different stress ranges.
- The specimens were fatigue loaded until the rupture of the steel plate.
- A sufficient number of tests were conducted in the experimental program, which consisted of testing 17 unreinforced specimens, 20 reinforced with 1 CFRP layer, and 11 reinforced with 3 CFRP layers.
- The steel used for the fabrication of the specimen is commonly used in bridge construction with yield stress equal to 388MPa and ultimate stress equal to 553MPa.
- Figure 6-1, the geometry of open-hole specimens is representative of holes in the tensile steel connection.

Figure 6-2 shows the fatigue data of un-retrofitted specimens is scattering between the Eurocode fatigue strength curve category 90 and 150. Reinforced specimens achieved a significant increase of fatigue life at lower stress levels where data points are converging toward a fatigue limit, especially for specimens with 3 layers of CFRP (CFRP3) which did not experience damage after 8 million cycles at $\Delta\sigma = 180MPa$. The appearance of the fatigue limit after the application of the CFRP is very important

for design purposes. Note that these results are very promising since most of the ancient metallic riveted assemblies are subjected to stress ranges below 150 MPa [38]. The extension of fatigue life tends to decrease while increasing the stress level. This can be clearly seen in Figure 6-3, where the fatigue life extension ratio (i.e. $Ratio = N_f(retrofitted)/N_f(bare)$) is decreasing while the stress level increases. Note $N_f(retrofitted)$ and $N_f(bare)$ used to calculate the $Ratio$ were obtained using linear fitting of the data. Figure 6-2 shows that the results of CFRP1 are overlapping the experimental data of bare steel specimens at $\Delta\sigma \geq 300MPa$. The reason behind this fact is due to the performance of the bonding at the steel-CFRP interface which tends to progressively weaken while increasing the stress level. This was also observed by Pastor et al [31] in similar tests on aluminum specimens. The fatigue data of the retrofitted specimens follow different trends with respect to the stress range level and a single line curve with slope $m=3$ as prescribed by fatigue design codes cannot provide a representative description of the fatigue data of retrofitted specimens. Therefore, comprehensive fatigue analysis of the data is needed to derive more adequate S-N curves.

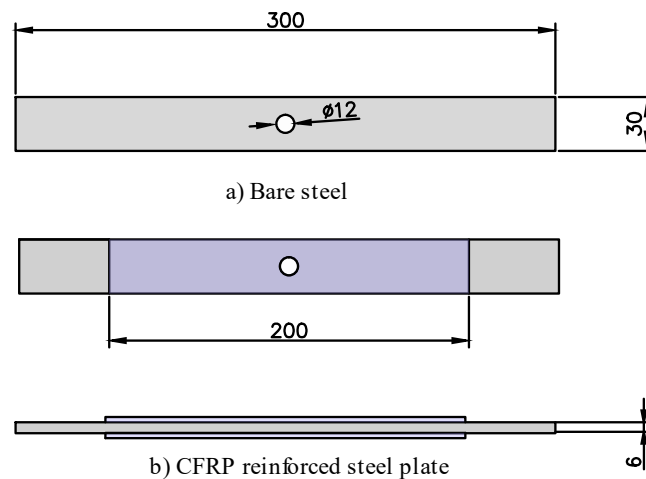


Figure 6-1: Open-hole steel specimens retrofitted with CFRP [32].

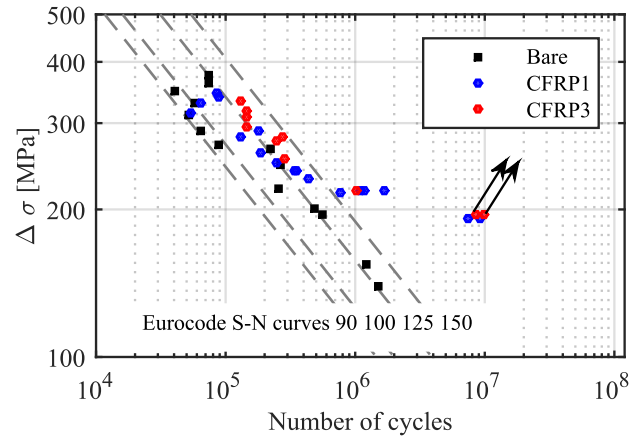


Figure 6-2: The Eurocode fatigue strength curves categories 90-150 and test data of open-hole specimens [32].

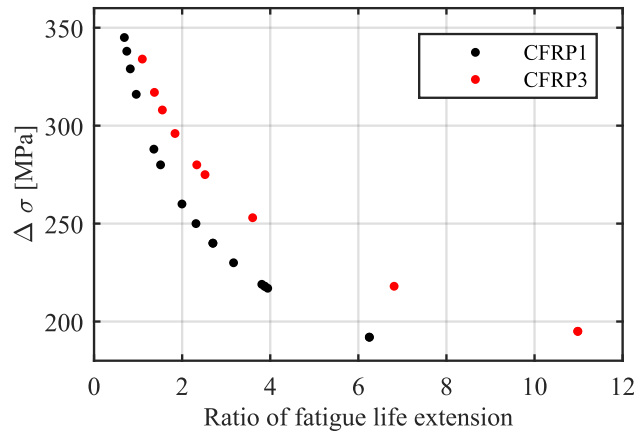


Figure 6-3: Ratio of increase of fatigue life versus the stress range, $\Delta\sigma$.

6.3 Derivation of P-S-N curves

Scattering is an inherent property of fatigue data due to several sources related to the material properties, surface quality, subsurface inclusions, and experimental errors. Since the fatigue life is random in nature, reliable failure prediction in fatigue problems is not possible without the use of statistical and probabilistic methods [37]. Unfortunately, the distribution function of the fatigue data cannot be derived based on physical arguments. In general, it is simply assumed, or adjusted to experimental data [39], [40]. To deal with fatigue scattering, several probabilistic models using different statistical distributions have been adopted in the literature [41]. In this study, four statistical distribution functions are adopted and compared to determine the best P-S-N curves for CFRP retrofitted specimens subjected to constant amplitude fatigue loading.

6.3.1 General regression models for fatigue S-N curves

There is an underlying linear relationship between the stress and the fatigue life in the log-log scale that can be modeled using Basquin's [42] linear relation given in Eq.1.

$$\log N_f = B - A \log \Delta\sigma \quad \text{Eq. 1}$$

Where, N_f is the number of cycles to failure, and $\Delta\sigma$ is the stress range. To further simplify the analysis Eq.1 is transformed to Eq.2, where X is $\log N_f$ and Y is $\log \Delta\sigma$.

$$X = B - AY \quad \text{Eq. 2}$$

The mean S-N curve is usually obtained by fitting the Eq.3 to the fatigue data using the maximum likelihood method, $\hat{a}$ can be obtained using Eq.4 and $\hat{b}$ using Eq.5. The caret sign “^” is used to indicate that they are estimates.

$$\hat{\mu}_x = \hat{b} - \hat{a} y \quad \text{Eq. 3}$$

$$\hat{a} = \frac{\sum_{i=1}^n (x_i - \bar{x})(y_i - \bar{y})}{\sum_{i=1}^n (y_i - \bar{y})^2} \quad \text{Eq. 4}$$

$$\hat{b} = \bar{x} - \hat{a} \bar{y} \quad \text{Eq. 5}$$

This linear relationship holds only true in the finite life range between certain limits of stress. The lower limit is the fatigue limit typically between 2×10^6 and 10^7 cycles and the upper limit is the boundary of the low cycle regime which is commonly taken as the allowable static design stress [43]. Censored data such as runouts or non-complete tests cannot be considered in the analysis of the data when using this method. This model is widely adopted in fatigue standards such as ISO12107 [34] and ASTM E739 [44]. To consider the variability of the data, these standards adopt lower boundary S-N curves assuming that the logarithm of the fatigue life follows a normal distribution. However, as stated previously the assumption of the distribution function is not based on physical arguments, so in the literature different distribution functions have been investigated [37], [41], [43], [45], [46]. The Extreme value type III distribution known as the Weibull distribution is commonly adopted for skewed data [37], [39], [41], [46], [47]. As an alternative Extreme value distribution function, the type I known as the Gumbel distribution is also adopted in several studies [41], [48]–[51].

A major issue in the derivation of S-N curves is the prediction of the fatigue life in the infinite life range when the fatigue data shows a fatigue limit. The methodology previously presented do not cover this region. ISO12107 [34] proposes to use a lower slope close to the horizontal line which needs to be defined using at least 6 tests. However, such data is usually unavailable in the literature for economic reasons. Castillo and Cantelli [37] proposed probabilistic models based on the Weibull and Gumbel distributions that can capture the finite and the infinite life ranges and requires a relatively lower number of tests in the infinite life range to calibrate the model.

6.3.2 ISO12107

ISO12107 [34] proposes a linear boundaries model that is relatively easy to implement in practice. The model, shown in Eq. 1, is modified to determine a lower fatigue limit using statistical methods assuming that the logarithm of the fatigue life follows a normal distribution with a constant variation. To obtain the P-S-N curves corresponding to a certain probability of failure, Eq.6 should be used.

$$\hat{x}_{(P,1-\alpha,v)} = \hat{b} - \hat{a}y - k_{(P,1-\alpha,v)} \sigma_x \sqrt{1 + \frac{1}{n} + \frac{(y - \bar{y})^2}{\sum_{i=1}^n (y_i - \bar{y})^2}} \quad \text{Eq. 6}$$

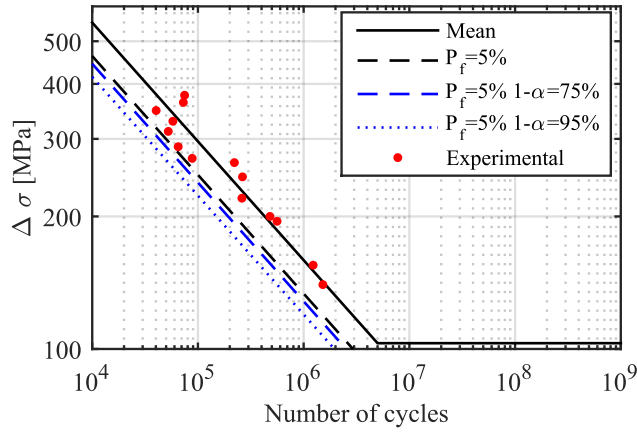
Where, $k_{(P,1-\alpha,v)}$ is the one-sided tolerance limit for normal distribution, and σ_x is the sample standard deviation of the logarithm of the fatigue life. The term inside the root is a correction factor that depends on the number of tests, when the number and range of the tests are large enough, the correction term is close to 1 and maybe neglected. Note the fatigue data points at $N_f > 5 \times 10^6$ cycles are not included in the fitting process. Such data are usually estimated by a second slope as in Eurocode 3 part 1-9 or by a horizontal line. Due to the lack of data, a horizontal line is assumed in this study.

Figure 6-4 shows the experimental data, the mean curves obtained using Eq. 3, and the lower limit curves obtained using Eq. 6 for the probability of failure $P_f = 5\%$ and confidence level of $1 - \alpha = 75\%$ and 95% are also presented. The lower limit curve $P_f = 5\%$ without the confidence interval was obtained by replacing the factor $k_{(P,1-\alpha,v)}$ by the inverse cumulative of the normal distribution. Figure 6-4 (a) shows that the mean curve fits well the experimental data of bare steel specimens and the lower curve ($P_f = 5\%$) provides a lower bound of the fatigue life. In Figure 6-4 (b), the fatigue data of CFRP1

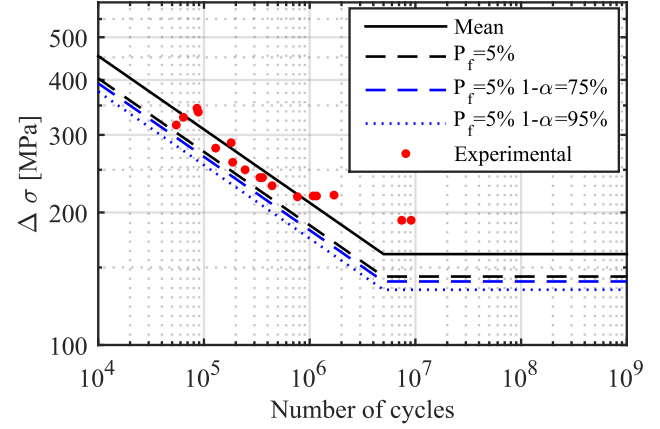
is showing a tendency of converging towards a fatigue limit at lower stress levels. The data tends to be skewed below the mean curve at medium cycles (i.e. $10^5 \leq N_f \leq 10^6$), and upper the mean curve at higher cycles. The fatigue life at high cycle fatigue is significantly underestimated by the mean curve. Figure 6-4 (c) illustrates the fatigue data of CFRP3 specimens. The mean curve fits reasonably well the data at the medium cycle regime. But it does not provide a good estimate of the fatigue life at high cycle fatigue. The $P_f = 5\%$ curves with confidence level of $1 - \alpha = 75\%$ are assumed to be the most suitable P-S-N curves as adopted in Eurocode 3 part 1-9, the parameters obtained in the fitting process are presented in Table 6-1. The fatigue curves for the probability of failure $P_f = 5\%$ and $1 - \alpha = 75\%$ for bare steel specimens, CFRP1, and CFRP3 are characterized by stress range $\Delta\sigma_c = 106 \text{ MPa}$, 162 MPa and 160 MPa at 2×10^6 cycles. Figure 6-4 (d), the comparison between the curves with $P_f = 5\%$ and $1 - \alpha = 75\%$ for the three types of specimens is presented. It can be seen that the curve for CFRP1 shows a lower slope than CFRP3 and the fatigue limit of CFRP1 is higher than CFRP3 which is not very reasonable. The CFRP1 curve intersects with the bare steel curve around $\Delta\sigma = 320 \text{ MPa}$ and provides lower fatigue life at $\Delta\sigma > 320 \text{ MPa}$ which is also not reasonable.

Table 6-1: The fitting parameters of the ISO method for P-S-N curves with $P_f = 5\%$ and $1 - \alpha = 75\%$.

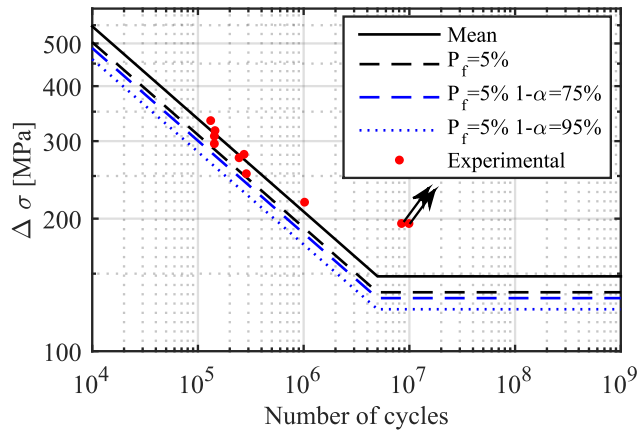
	Bare steel	Retrofitted with 1 layer of CFRP (CFRP1)	Retrofitted with 3 layers of CFRP (CFRP3)
No. of specimens	17	20	11
Standard deviation, σ	0.162	0.256	0.099
$\hat{b}$	14.16	19.93	17.07
$\hat{a}$	-3.7	-6.0	-4.75
$k_{(P,1-\alpha,v)}$	3.885	2.01	2.25



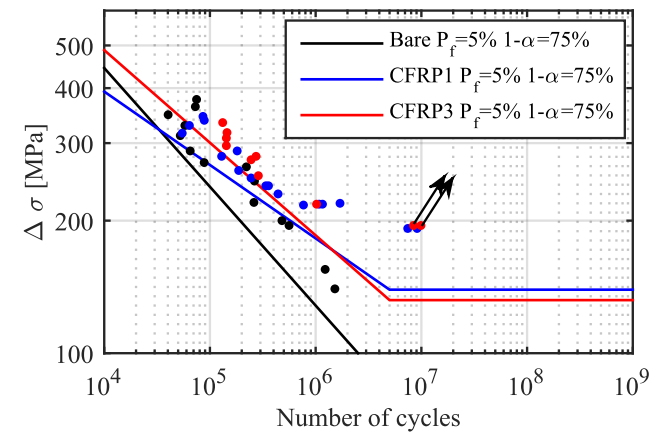
(a) Bare steel specimens



(b) Specimens reinforced with 1 layer of CFRP



(c) Specimens reinforced with 3 layers of CFRP



(d) P-S-N curves with $P_f = 5\%$ and $1 - \alpha = 75\%$.

Figure 6-4: P-S-N curves obtained using IS12107 method.

6.3.3 Weibull model

The Weibull distribution is the most recommended distribution to correlate skewed fatigue data [37], [45]. International codes such as ISO12107 [34] and ASTM E739 [44] that adopted the normal distribution, also recommend the use of Weibull statistical distribution to model the variations in fatigue data. The cumulative distribution function (CDF) of the two parameters Weibull distribution used in this study, is presented in Eq. 7.

$$P(N_{norm}) = 1 - e^{-\left[\frac{N_{norm}}{a_w}\right]^{\beta_w}} \quad \text{Eq. 7}$$

Where, N_{norm} is the normalized number of cycles, which is estimated using Eq.8.

$$N_{norm} = \frac{N_f}{\bar{N}} \quad \text{Eq. 8}$$

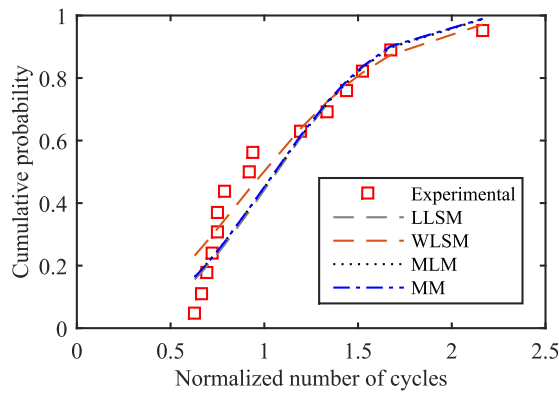
Where, $\bar{N}$ is the estimated number of cycles to failure obtained using Eq.3; α_w is the Weibull scale parameter; and β_w is the Weibull shape parameter. The Weibull parameters are obtained by fitting the cumulative probability of the experimental data with Bernard median rank and the cumulative probability of the Weibull distribution. The maximum likelihood method (MLM) is widely adopted to fit the Weibull distribution. However, for the sake of verification and following the recommendations of Barbosa et al. [52], the parameters were estimated with three additional other methods: the linear least square method (LSM), the weighted linear least square method (WLSM), and the method of moments (MM). More details about the implementation of each method can be found in [45], [52]. To show that the MLM method provides a reasonable fitting, a graphical comparison is presented in Figure 6-5. Note Figure 6-5 (b) shows that the Weibull distribution fits poorly the variation of CFRP1, therefore, another extreme value distribution such as the Gumbel distribution is worth investigating. Table 1 presents the fitted parameters using different methods.

The P-S-N curves were determined using Eq.8, as was proposed by Júnior et al. [53].

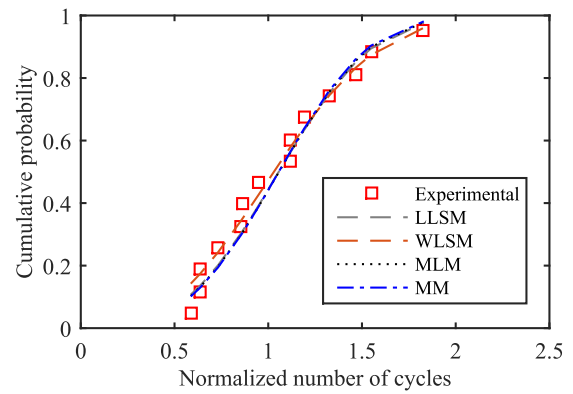
$$\Delta\sigma = A_w - B_w \left[\log \left(\frac{N_f}{\beta_w [-\ln(1-P)]^{\frac{1}{\alpha_w}}} \right) \right] \quad \text{Eq. 9}$$

Where A_w and B_w are parameters obtained by fitting the experimental data using the linear least square method. They are equal to $A_w = -\frac{\hat{b}}{\hat{a}}$ and $B_w = -\frac{1}{\hat{a}}$ using the parameters previously presented in Table 6-1. Figure 6-6 illustrates the experimental fatigue data and the P-S-N curves corresponding to $P_f = 50\%$ and $P_f = 5\%$. The P-S-N curves with $P_f = 5\%$ and confidence level of $1 - \alpha = 95\%$ and $1 - \alpha = 75\%$ are also presented. Figure 6-6 (a) shows that the mean curve correlates well with the experimental data of bare steel specimens and the curves corresponding to $P_f = 5\%$ represent conservative limits to the experimental data. Figure 6-6 (b) shows that the fatigue data of CFRP1 specimens are skewed below the mean fatigue curve at the medium cycle regime whereas at higher cycles the mean curve is quite conservative. The lower limit curve corresponding to $P_f = 5\%$ shows a conservative estimate of the data overall, and very conservative at high cycle regime. Figure 6-6 (c)

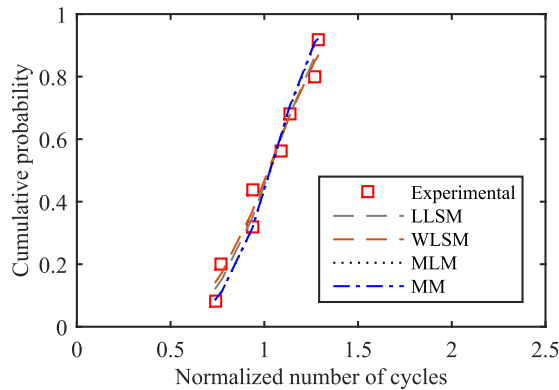
shows the experimental data of CFRP3 specimens, where the mean curve correlates the experimental data at medium cycles, but it provides a very conservative estimate at the high cycle regime. The fatigue curves with $P_f = 5\%$ and $1 - \alpha = 75\%$ for bare steel specimens, CFRP1 specimens and CFRP3 specimens are characterized by $\Delta\sigma_c = 97$ MPa, 150MPa, and 156MPa at 2×10^6 cycles. Figure 6-6 (d) illustrates the comparison of the P-S-N curves with $P_f = 5\%$ and $1 - \alpha = 75\%$ for the three types of specimens, where it can be seen that the curve for CFRP1 shows a lower slope than the two other curves. The CFRP1 and CFRP3 curves show similar fatigue limit, this is more reasonable than the curves obtained in ISO. However, the CFRP1 curve intersects with the bare steel curve around $\Delta\sigma = 300$ MPa and provides lower fatigue life at $\Delta\sigma > 300$ MPa which is not reasonable.



(a) Bare steel specimens.



(b) Specimens retrofitted with 1 layer of CFRP (CFRP1).

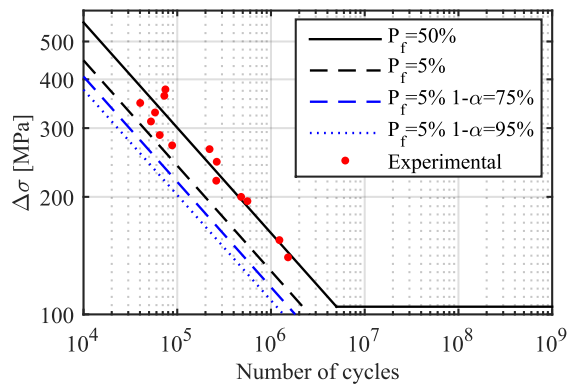


(c) Specimens retrofitted with 3 layers of CFRP (CFRP3).

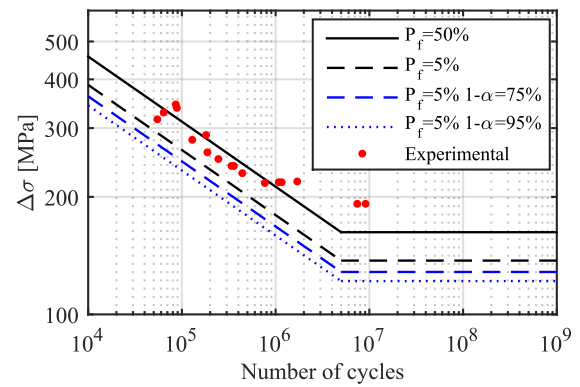
Figure 6-5: Cumulative probability based on the Weibull distribution.

Table 6-2: Fitting parameters of the two parameters Weibull model.

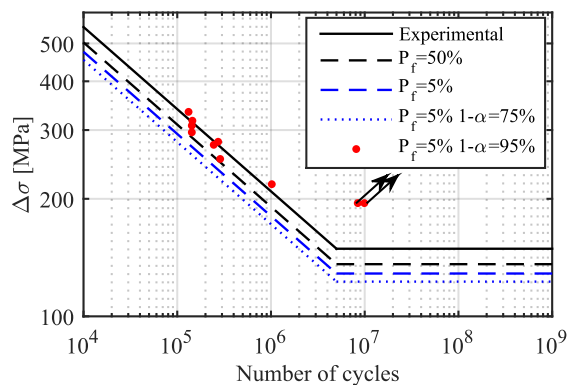
Specimens	Method	LSM	WLSM	MLM	MM
Bare steel specimens	α_w	3.0637	2.4347	2.8513	2.9433
	β_w	1.1928	1.1659	1.1959	1.1923
Retrofitted with 1 layer of CFRP (CFRP1)	α_w	2.6644	2.1025	2.5940	2.6215
	β_w	1.2232	1.1840	1.2180	1.2135
Retrofitted with 3 layers of CFRP (CFRP3)	α_w	2.1265	1.9798	2.4520	2.3878
	β_w	1.2810	1.2813	1.2678	1.2632



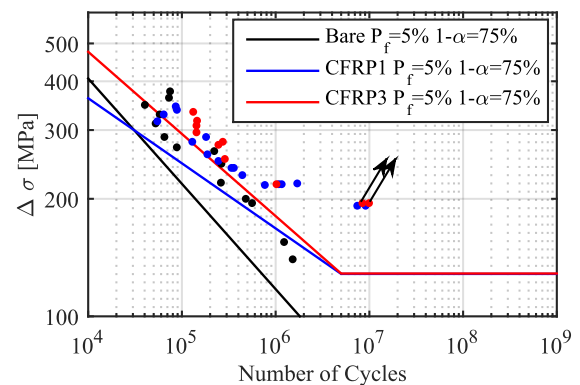
(a) Bare steel specimens.



(b) Specimens retrofitted with 1 layer of CFRP (CFRP1).



(c) Specimens retrofitted with 3 layers of CFRP (CFRP3).



(d) Comparison between the P-S-N curves for the three types of specimens.

Figure 6-6: P-S-N curves derived using the two-parameter Weibull distribution model.

6.3.4 Gumbel model

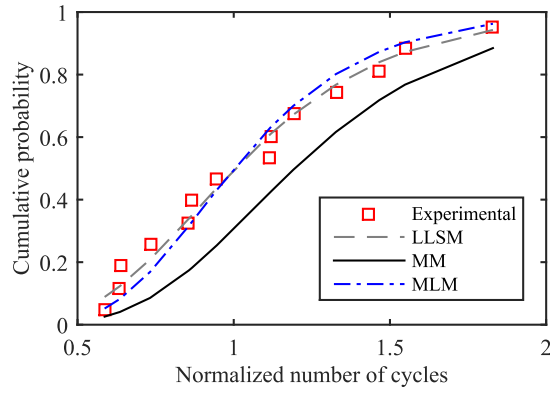
As it was shown in the previous section, the Weibull distribution did not fit the data of CFRP1 specimens. So, it is worth investigating other extreme value distributions such as the Gumbel distribution. The CDF of the Gumbel distribution used in this study is presented in Eq.10.

$$P(N_{norm}) = e^{-e^{-\frac{N_{norm}-\alpha_G}{\beta_G}}} \quad \text{Eq. 10}$$

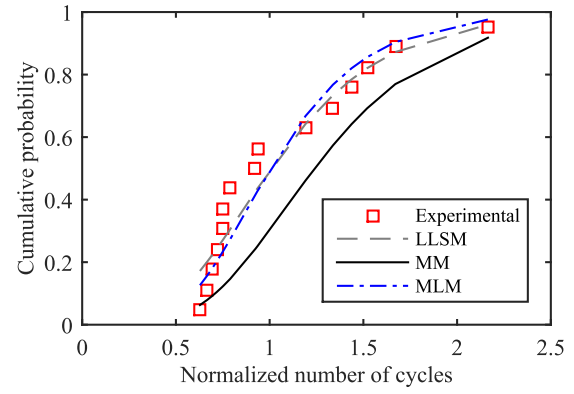
Where, α_G and β_G are the location and shape parameters of the probability function. N_{norm} is the normalized fatigue life obtained using Eq.8. The shape and location parameters were estimated by fitting the cumulative probability of the experimental data using Bernard median rank and Eq.10. The LSM, the MM, and the MLM methods were used in the fitting process. Figure 6-7 shows the comparison of the fittings using the three different methods. According to the graphical examination. The MLM method provides very poor fitting. It is worth noting that there is often evidence that the MLM method does not perform well in fitting the Gumbel distribution, especially, in the case of small samples [54]. The LSM and MM provide more reasonable fittings. For the sake of simplicity, the LSM method is adopted for the derivation of the P-S-N curves. The parameters derived in the fitting process using different methods are presented in Table 6-3. Figure 6-7 (b) shows that the data of CFRP1 specimens fits also poorly the Gumbel distribution which is very similar to the Weibull distribution.

Table 6-3: Fitting parameters of the Gumbel distribution.

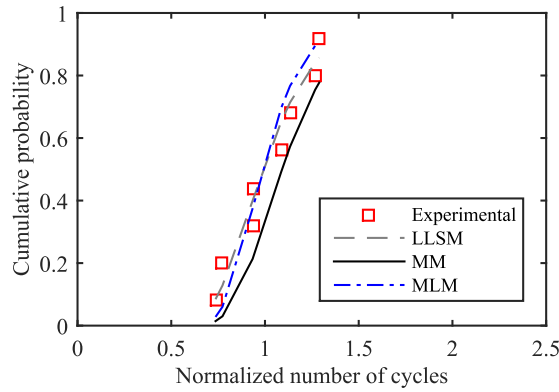
Specimens	Method	LSM	MLM	MM
Bare steel specimens	α_w	0.8825	1.0612	0.8965
	β_w	0.3343	0.3659	0.2853
Retrofitted with 1layer CFRP (CFRP1)	α_w	0.81650	1.1780	0.8520
	β_w	0.669	0.7244	0.5648
Retrofitted with 3 layers CFRP (CFRP3)	α_w	0.9168	1.0193	0.9311
	β_w	0.1995	0.1960	0.1528



(a) Bare steel specimens.



(b) Specimens retrofitted with 1 layer of CFRP (CFRP1).



(c) Specimens retrofitted with 3 layers of CFRP (CFRP3).

Figure 6-7: Fitting of the Gumbel distribution function.

The P-S-N curves were calculated using Eq.12 which was determined by replacing the inverse CDF of Gumbel distribution shown in Eq.11 and Eq.8 in Eq.1.

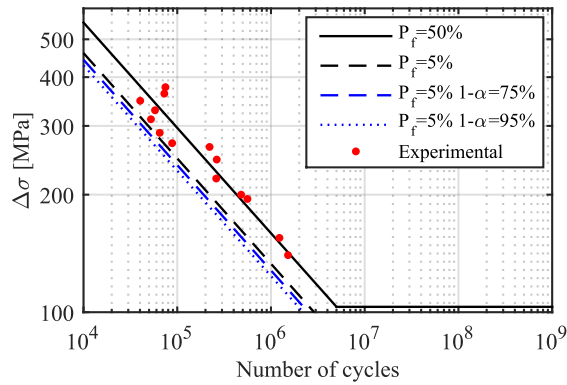
$$N_{\text{norm}} = \alpha_G - \beta_G (\ln(-\ln P)) \quad \text{Eq. 11}$$

$$\Delta\sigma = A_G - B_G \left[\log \left(\frac{N_f}{\alpha_G - \beta_G (\ln(-\ln P))} \right) \right] \quad \text{Eq. 12}$$

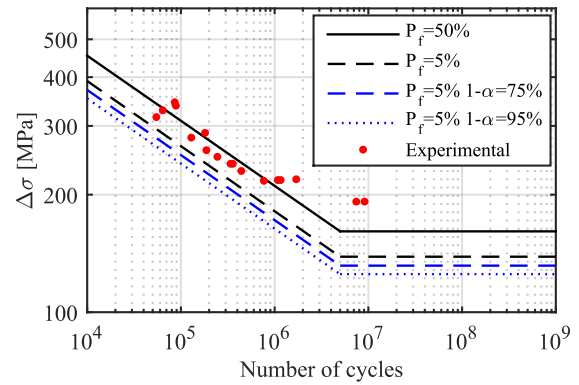
Where A_G and B_G are parameters obtained by fitting the experimental data using the linear least square method. They are exactly equal to A_W and B_W obtained in the previous section.

illustrates the fatigue data of bare steel specimens and retrofitted specimens and the P-S-N curves with $p_f = 50\%$ and $p_f = 5\%$. The P-S-N curves obtained using the Gumbel model show very similar trends to the ones obtained using the Weibull distribution. Thus, all the observations previously reported for the Weibull P-S-N curves also apply for the Gumbel P-S-N curves shown in Figure 6-8, except, in

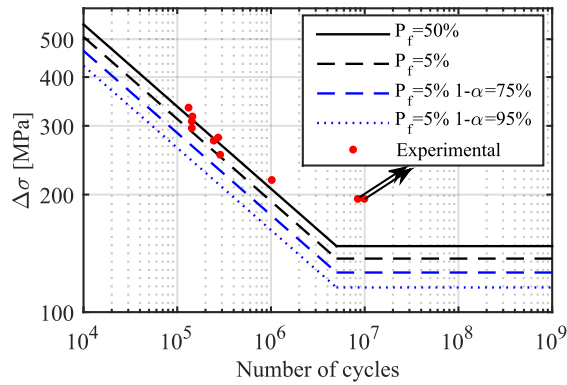
Figure 6-8 (d) where the CFRP1 curve shows a higher fatigue limit than CFRP3 which is similar to the ISO curves and the CFRP1 curve intersects with the bare curve at a lower stress range around $\Delta\sigma = 280\text{MPa}$. The fatigue curves for the probability of failure $P_f = 5\%$ and confidence level of 75% for bare, CFRP1, and CFRP3 specimens are characterized by $\Delta\sigma_c = 106\text{ MPa}$, 153MPa , and 153MPa at 2×10^6 cycles.



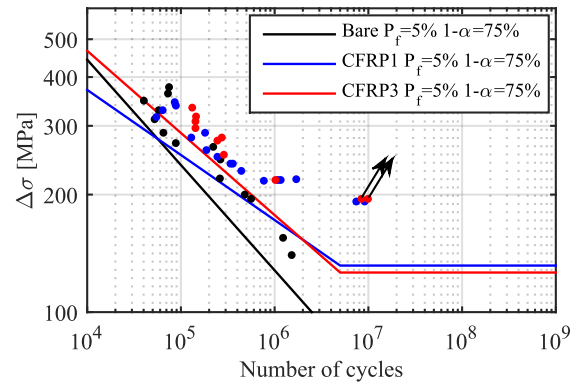
(a) Bare steel specimens.



(b) Specimens retrofitted with 1 layer of CFRP (CFRP1).



(c) Specimens retrofitted with 3 layers of CFRP (CFRP3).



(d) Comparison between the P-S-N curves for the three types of specimens.

Figure 6-8: P-S-N curves derived using the Gumbel distribution.

6.3.5 Castillo and Fernández-Canteli model (CFC)

The CFC model [37] presented in this section was derived assuming the fatigue data follows the three-parameter Weibull distribution and dimensional analysis. This model is recommended for medium to

high cycle regime. The model can capture well the transitions between low and high cycle fatigue. Censored fatigue data can be included in the calibration of the model using the E-M algorithm. The cumulative probability distribution function of the model is presented in Eq.13. The P-S-N curves could be estimated using Eq.14.

$$F(\log N_f; \log \Delta\sigma) = p = 1 - \exp \left\{ - \left[\frac{(\log N_f - B)(\log \Delta\sigma - C) - \lambda}{\delta} \right]^\beta \right\} \quad \text{Eq. 13}$$

$$(\log(N_f) - B) \cdot (\log(\Delta\sigma) - C) \geq \lambda$$

$$\Delta\sigma = \exp \left(C + \frac{\lambda + \delta(-\log(1 - P)^{1/\beta})}{\log(N_f) - B} \right) \quad \text{Eq. 14}$$

Where: N_f is the number of cycles to failure; $\Delta\sigma$ is the stress range; λ , β and δ are the non-dimensional Weibull parameters, where λ is the Weibull location parameter, β is the Weibull shape parameter and δ is the Weibull scale parameter; $C = \log(\Delta\sigma_0)$, where $\Delta\sigma_0$ defines the endurance limit, and $B = \log(N_0)$, which is the threshold value of the lifetime. The model is calibrated in two steps: first by minimizing the three parameters nonlinear equation to determine B and C. then, the data points can be pooled together to obtain the dimensional parameter V_i , shown in Eq.16, which in turn is assumed to follow a three parameters Weibull distribution shown in Eq.7. The fitting of the three parameters Weibull distribution can be obtained by several methods among them are the MLM, probability-weighted moments[55], the Castillo-Hadi [56]. In this study, the parameters of the model were calibrated using ProFatigue software [57]. The confidence level can be obtained using the bootstrapping method. However, for the sake of simplicity and convergence problems the confidence level was not considered. Table 6-4 presents the parameters for each set of specimens.

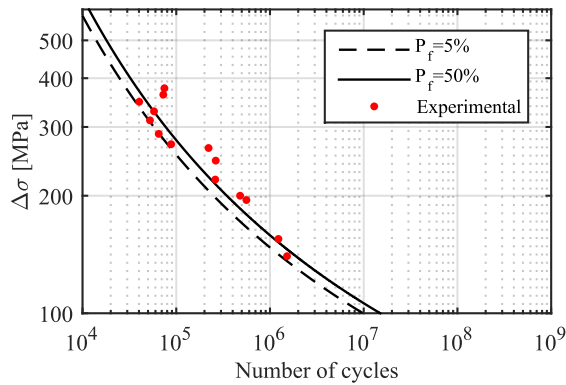
$$Q = \sum_{i=1}^n \left(\log N_i - B - \frac{\mu}{\log \Delta\sigma_i - C} \right)^2 \quad \text{Eq. 15}$$

$$V_i = (\log N_i - B)(\log \Delta\sigma_i - C) \quad \text{Eq. 16}$$

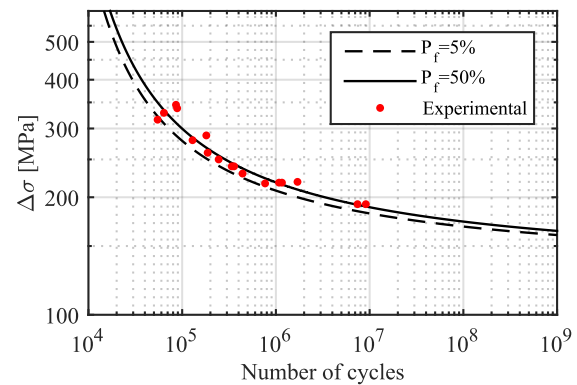
$$P(V) = 1 - e^{-\left[\frac{V-\lambda}{\delta}\right]^\beta} \quad \text{Eq. 17}$$

Figure 6-9 illustrates the P-S-N curves calculated using the CFC model for bare steel and retrofitted specimens. (a) shows that the curves for fatigue data of bare steel specimens are not exactly linear. The

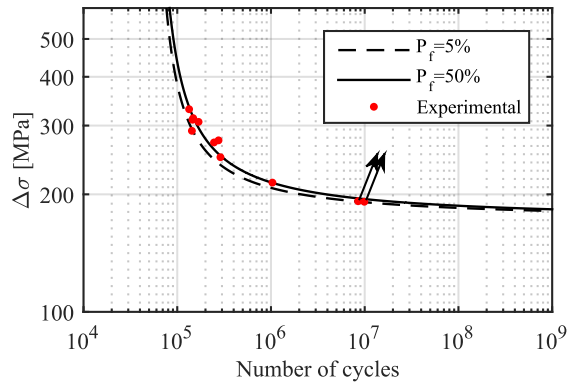
$P_f = 50\%$ curve correlates well the experimental data and the $P_f = 5\%$ curve presents a lower limit that is not conservative for all data points. Figure 6-9 (b) shows that the P-S-N curves for CFRP1 specimens have a nonlinear shape that follows the different trends of the experimental data points from medium cycle to high cycle regime. The lower curve corresponding to $P_f = 5\%$ does not provide a conservative prediction for all the data points. Figure 6-9 (c) shows that the P-S-N curves capture well the trends of the experimental data as well as the fatigue limit where the curve is showing an asymptote converging towards the runouts. According to the data of Table 6-4, the fatigue limit for CFRP1 and CFRP3 specimens are 126MPa and 170MPa , respectively. Figure 6-9 (d) compares the P-S-N curves with $P_f = 5\%$ for the three types of specimens. It can be seen that in contrary to the previous linear models, the CFC model captured well the behavior of the CFRP1 data points, where the model does not underestimate the fatigue life at higher stress levels and shows a reasonable distinction between the fatigue limit for CFRP1 and CFRP3 specimens.



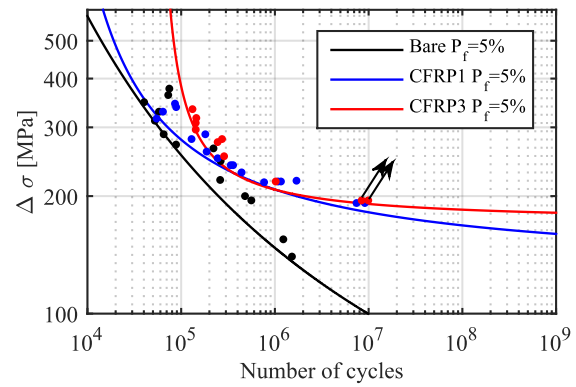
(a) Bare steel specimens.



(b) Specimens retrofitted with one layer of CFRP (CFRP1).



(c) Specimens retrofitted with three layers of CFRP (CFRP3).



(a) Comparison between the P-S-N curves for the three types of specimens.

Figure 6-9: P-S-N curves derived using the CFC model.

Table 6-4: Parameters of the CFC model.

Specimens	B	C	N_0	$\Delta\sigma_0$	β	δ	λ
Bare steel	0	2.26	1	9.58	1.04	1.54	37.67
CFRP1	7.61	4.84	2.01×10^3	126.47	2	0.43	2.99
CFRP3	10.77	5.14	4.75×10^4	170.7	2.65	0.18	0.54

6.4 DISCUSSION

6.4.1 Comparison of the P-S-N curves

This section presents a comparison of the P-S-N curves generated from the four models presented previously and the Eurocode S-N curves. Table 6-5 summarizes the key parameters of the P-S-N curves from Eurocode 3 and the ones derived in this study that will be included in the comparison.

Figure 6-10 shows the P-S-N curves corresponding to a probability of $P_f = 5\%$ for the bare specimens and the Eurocode S-N curve category 90. The ISO and Gumbel models provide roughly similar curves that are less conservative than the Weibull curve. The P-S-N curve obtained using the CFC is not linear and very close to the experimental data points, especially at higher stress. Due to the limited number of tests. A lower probability of failure when using the CFC model is more recommended. The Eurocode S-N curve category 90 seems to be a suitable conservative design curve, it limits well the data, especially at higher stress levels. Figure 6-11 shows the experimental data of CFRP1 specimens, the P-S-N curves, and the Eurocode S-N curve category 100. In general, the linear curves obtained using the ISO, Weibull,

and Gumbel poorly fit the experimental data. The ISO method provides a less conservative linear curve. The Gumbel and the Weibull models provide quite similar curves, with the Gumbel being slightly less conservative. The linear curves give reasonable conservative predictions of the experimental data at higher and medium stress levels but very conservative for low-stress levels. The fatigue data of CFRP retrofitted specimens do not follow a straight line but nonlinear trends that depend on the stress level as discussed previously in this chapter. Maybe the bilinear model with fitted data at high cycle range as recommended by ISO12107 could predict better the fatigue life at higher cycles. However, due to the limited experimental data at this range, this could not be done. The CFC model provides a nonlinear P-S-N curve that can follow the different trends of the experimental data. Clearly, the CFC model predicts well the fatigue at different stress levels especially at the transition between medium and high cycle ranges. Furthermore, a prediction of the fatigue limit at high cycle range $\Delta\sigma_0$ could be defined as shown in Table 6-5.

Figure 6-12 shows the fatigue data of CFRP3 specimens, the P-S-N curves obtained using the models presented previously, and the Eurocode design curve category 125. It can be seen that Eurocode S-N curve category 125 estimates well the fatigue data points at medium cycle range, but very conservative at high cycle range. The ISO, Weibull, and the Gumbel models provide good lower boundary lines at a medium cycle regime. The ISO curve provides a less conservative prediction compared to the Weibull and the Gumbel curves. The CFC model provides a very good fitting of the experimental data at the different stages including the two runouts. Thus, a fatigue limit could be determined at $\Delta\sigma_0 = 170\text{MPa}$, as shown in Table 6-5.

Table 6-5: Summary of the parameters for the P-S-N curves.

Probabilistic Models	Specimen type	Bare steel	Retrofitted with 1 layer of CFRP (CFRP1)	Retrofitted with 3 layers of CFRP (CFRP3)
EC3	Inverse slope, m	3	3	3
	Confidence level	0.75	0.75	0.75
	Probability of failure [%]	5	5	5
	Detail category, $\Delta\sigma_c$ [MPa]	90	100	125
ISO12107	Inverse slope, m	3.7	6	4.75

	Confidence level	0.75	0.75	0.75
	Probability of failure [%]	5	5	5
	Detail category, $\Delta\sigma_c$ [MPa]	107	163	160
Weibull	Estimation Method	MLM	MLM	MLM
	α_w	2.8513	2.5940	2.4520
	β_w	1.1959	1.2180	1.2678
	Probability of failure [%]	5	5	5
	Inverse slope, m	3.7	6	4.75
	Confidence level	0.75	0.75	0.75
	Detail category, $\Delta\sigma_c$ [MPa]	97	150	156
Gumbel	Estimation Method	LSM	LSM	LSM
	α_G	0.8825	0.8165	0.9168
	β_G	0.3343	0.6690	0.1995
	Probability of failure [%]	5	5	5
	Inverse slope, m	3.7	6	4.75
	Confidence level	0.75	0.75	0.75
	Detail category, $\Delta\sigma_c$ [MPa]	106	153	153
CFC model	β	1.04	2	2.65
	δ	1.54	0.43	0.18
	λ	37.67	2.99	0.54
	$\Delta\sigma_0$	9.58	126.47	170.7
	N_0	1	2×10^3	4.7×10^4

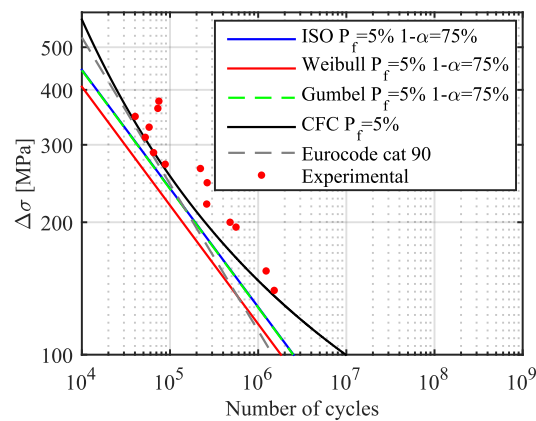


Figure 6-10: Comparison between the probabilistic fatigue models using bare steel specimens.

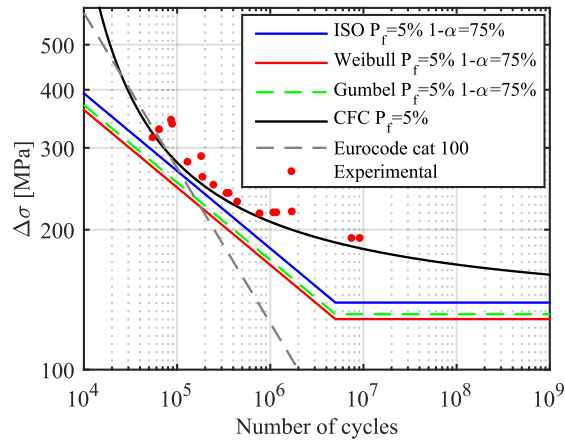


Figure 6-11: Comparison of the P-S-N curves for retrofitted specimens with 1 layer of CFRP.

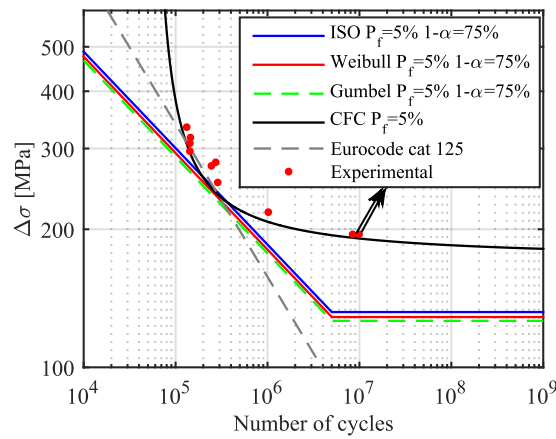


Figure 6-12: Comparison of P-S-N curves for retrofitted specimens with 3 layers of CFRP.

6.4.2 Proposal of P-S-N curves for retrofitted specimens

As it was shown in the previous section, the Eurocode S-N curves and the linear curves in log-log scale derived by the ISO, the Weibull, the Gumbel models fail to provide a good prediction of the fatigue life for CFRP retrofitted specimens especially at lower stress level (i.e. $180\text{MPa} \leq \Delta\sigma < 250\text{MPa}$). At this stress range, the CFRP retrofit provides its optimum performance and is very relevant to practical cases as discussed previously in this study. Therefore, a nonlinear model in the log-log scale such as the CFC model is required to model the fatigue behavior of the CFRP retrofitted specimens. The CFC model fitted very well the trends of the experimental data of retrofitted specimens and the model

provided a good estimate of the fatigue endurance limit which is very important for design purposes. However, due to the limited number of experimental data, the CFC model provided P-S-N curves that are not conservative enough. Therefore, it is recommended to adopt more conservative curves with a lower probability of failure. The curves with $P_f = 1\%$ are proposed for CFRP1 and CFRP3 specimens, as shown in Figure 6-13. The equations to derive P-S-N curves for CFRP1 and CFRP 3specimens are presented in Eq. 18 and Eq. 19, respectively.

$$\Delta\sigma = \exp \left(4.84 + \frac{2.99 + 0.43(-\log(1 - P)^{1/2.65})}{\log(N_f) - 7.61} \right) \quad \text{Eq. 18}$$

$$\Delta\sigma = \exp \left(5.14 + \frac{0.54 + 0.18(-\log(1 - P)^{1/2.65})}{\log(N_f) - 10.77} \right) \quad \text{Eq. 19}$$

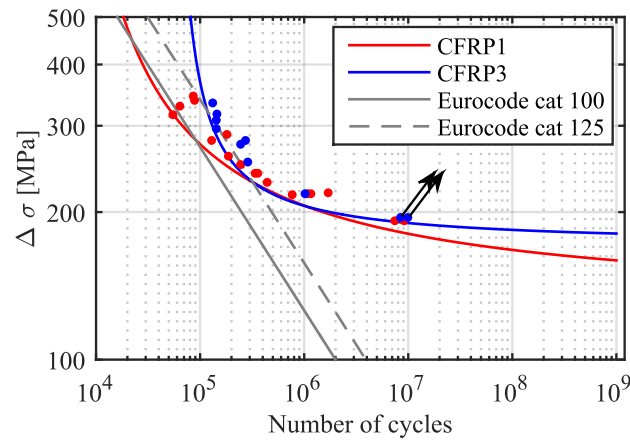


Figure 6-13: Fatigue resistance curves for retrofitted specimens with CFRP.

6.5 SUMMARY

Currently, the retrofit of metallic bridges is a major issue due to the high number of aging bridges. There is a high interest in applying CFRP in metallic old bridges to reduce stress concentration and prevent crack initiation. However, little experimental testing is published in the literature on this topic. The experimental data on open-hole specimens provided by Wang et al. [32] showed very promising results for retrofitting metallic bridges since a fatigue limit was shown at $\Delta\sigma = 120\text{MPa}$ for steel specimens retrofitted with 1 layer of CFRP (CFRP1) and $\Delta\sigma = 170\text{MPa}$ for specimens retrofitted with 3 layers

of CFRP (CFRP3). Usually, the stress range in ancient riveted metallic is below 150MPa. Therefore, the application of CFRP at local connections should provide an important extension of fatigue life.

This chapter focused on the analysis of fatigue data of non-damaged steel specimens reinforced with CFRP and the derivation of P-S-N curves. Firstly, experimental data of open-hole specimens were collected from the literature. A discussion on the behavior of CFRP retrofitted open-hole specimens subjected to fatigue loading was provided. In general, the CFRP retrofit was very efficient in extending the fatigue life especially at stress range $\Delta\sigma < 220MPa$, where the fatigue data points converge towards a fatigue limit which was not present for bare steel open-hole specimens even at $\Delta\sigma = 140MPa$. However, the extension of fatigue life tends to decrease while the stress level increases. This is mainly due to the performance of the bonding at the steel-CFRP interface which weakens at high-stress levels. Therefore, the fatigue data of retrofitted specimens showed different trends depending on the stress level and followed a nonlinear curve in the log-log scale.

The derivation of fatigue resistance curves requires the use of statistical methods, due to the high scatter of fatigue data. Since the distribution function of fatigue data is unknown, P-S-N curves were derived assuming different statistical distribution functions, namely: the Normal distribution, the two-parameter Weibull distribution, the Gumbel distribution, and the three parameters Weibull distribution. The Normal distribution was applied using the method proposed in ISO12107. The cumulative functions of the two parameters Weibull distribution and Gumbel distribution were combined with the linear Basquin model to derive P-S-N curves. The three parameters weibull distribution was applied using the model developed by Castillo & Fernández-Canteli.

The results showed that the linear models derived using the Normal distribution, the two parameters Weibull distribution and the Gumbel distribution could not capture the behavior of the experimental data of CFRP retrofitted specimens. Whereas, the CFC model provided nonlinear curves that fitted very well the different trends of the experimental data for specimens retrofitted with CFRP. The transition between the medium and high cycle regimes was well captured by the CFC model. Besides, a prediction of the fatigue limit, which is very important for design purposes, could be provided by the use of the CFC model. However, the CFC model provided P-S-N curves which were not conservative enough.

Therefore, P-S-N curves with a probability of failure, as low as $P_f = 1\%$, are recommended for CFRP retrofitted open-hole steel specimens.

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CHAPTER 7: An approach for predicting the fatigue life of CFRP retrofitted metallic structural details

7 An approach for predicting the fatigue life of CFRP retrofitted metallic structural details

Abstract

This chapter presents a new approach to estimate the fatigue life of metallic details retrofitted with bonded CFRP patch. Existing phenomenological and analytical methods to predict the fatigue life of metallic structural details retrofitted with bonded CFRP patch do not consider the cyclic degradation of the composite patch. As for all materials, the composite patch (CFRP and adhesive) will experience degradation during fatigue cyclic loading. In this regard, a new approach that takes into account the fatigue cyclic degradation of the composite patch is proposed in this study by introducing in the fatigue damage accumulation model of the metallic element, a cycle-dependent stiffness degradation parameter of the composite patch. The stiffness degradation parameter is obtained using a fatigue stiffness degradation model calibrated from experimental fatigue data of CFRP adhesively bonded double strap joints. Experimental data available in the literature of central hole aluminium plates bonded with CFRP are utilized for validation of the approach.

7.1 Introduction

Fatigue is recognized as one of the most common causes of mechanical failures in metallic structures [1]–[4]. Adhesively bonded carbon fibre reinforced polymers (CFRP) patched on metallic components reduce the average stress in the original structure and can significantly increase the fatigue life. This technology has been implemented to repair aeronautical structures for more than three decades [5]. In the last two decades, considerable research efforts have been devoted to implementing this reinforcement technique in civil engineering steel and aluminium structures [6]–[15]. However, the use of this reinforcing method in practice is still in its early stages. In order to widely implement this technique, simple and accurate design procedures are required to be developed and implemented in design guidelines.

In the context of fatigue, CFRP patches are applied to achieve two objectives: crack growth retardation of cracked structural details or retrofitting non-cracked elements as a prevention measure. The fatigue crack growth retardation using CFRP patches received most of the research attention in the last two

decades. A detailed review of past experimental tests, as well as an effective method for modelling fatigue crack propagation of CFRP repaired structural details can be found in Mohabeddine et al. [9]. Little research attention was given to retrofitting non-cracked elements [16], [17] which does not address an important demand from governmental agencies responsible for the maintenance of the metallic infrastructures and building owners. Most of the applications of CFRP in practice concerned with fatigue problems have been implemented in aging metallic bridges as a prevention measure to delay crack initiation (e.g. [18]–[22]). However, experimental tests were required to be conducted before the implementation of the CFRP retrofits in the aforementioned case studies since current fatigue codes for steel or aluminium structures, such as Part 1-9 of Eurocode 3 [23], BS7910 [24], and Part 1-3 of Eurocode 9 [25], do not provide design guidance for CFRP retrofits. Specialized standards for the design and construction of externally bonded CFRP, such as CNR-DT 202 [26] and CIRIA 595 [27], acknowledge their limitations in the case of fatigue loading and suggest limiting the adhesive capacity to 20-30% of the static failure strength.

The bonded composite patch shares the average stress with the metallic element. If the reduced stress in the metallic element is evaluated, the fatigue life can be predicted using code-based S-N curves. Hu et al. [28] adopted this method and provided S-N curves for patched metallic specimens. However, the authors did not consider that the composite patch can degrade under fatigue loading, which is a limitation of their work. Most studies in the literature similarly did not consider the effect of fatigue loading on the composite patch [29]–[31]. As for all materials, the composite patch is susceptible to fatigue damage which depends mainly on the stress level, the stress state, and the fatigue resistance of the adhesive-CFRP patch. Hussein et al. [32] experimentally observed a fatigue bond failure in a CFRP patched cracked steel specimen subjected to fatigue loading and highlighted the importance of considering the fatigue performance of the adhesive bond. Pastor et al. [33] tested two sets of non-cracked aluminium plates patched with CFRP under fatigue loading with two stress ranges, namely $\Delta\sigma = 298 \text{ MPa}$ and $\Delta\sigma = 349 \text{ MPa}$. In the first stress range, the fatigue lifetime of patched specimens over unpatched specimens was increased by a factor of 7.2 whereas, in the second stress range, the fatigue lifetime was increased only by 1.5. The less efficiency of the CFRP reinforcement in the second

case ($\Delta\sigma = 349 \text{ MPa}$) was mainly due to the fatigue weakening of the adhesive layer at higher stress level. In another study, Pastor et al. [31] investigated the effect of bonding composite patches on non-cracked aluminium plates featuring a hole in the centre subjected to fatigue loading. The results showed a significant extension of the lifetime for patched over un-patched specimens except for higher loads, where the adhesive layer progressively failed under fatigue. Mohabeddine et al. [16] showed that the fatigue life extension ratio caused by the application of a CFRP patch on steel non-cracked specimens is not constant and decreases as the stress level increases due to the fatigue sensitivity of the composite patch at higher stress demands. Recently, Doroudi et al. [34] developed a numerical procedure to predict the fatigue life of cracked steel plates considering the debonding propagation of the CFRP-to-steel bonded interface using a cyclic plasticity model. Alternatively, the most common method to simulate the damage in the CFRP-to-steel adhesive interface is the cohesive zone modelling (CZM) technique that can be used for static and fatigue cases [35].

The existing methods that can consider the fatigue degradation of the composite patch on CFRP reinforced metallic specimens subjected to fatigue loading are all based on advanced numerical simulations (e.g. cyclic plasticity [34] and CZM [35]). Although numerical simulations can provide a very good insight into the complex behavior of the system for fatigue problems, it is usually computationally expensive. For design purposes especially for the construction industry, simplified analytical methods need to be developed. To the best knowledge of the authors, there are no such methods available in the literature that can estimate the fatigue life of CFRP reinforced specimens considering the fatigue degradation of the composite patch.

For the first time, this study proposes an approach to estimate the fatigue life of CFRP reinforced metallic structural details based on the cumulative fatigue damage concept. The reduced stress in the metallic structural detail is evaluated after each cycle considering the fatigue degradation of the composite patch by introducing a cycle-dependent stiffness degradation parameter in the calculation of the reduced stress. The fatigue stiffness degradation model of the composite patch, calibrated from fatigue experimental data on double strap joint, is used to evaluate the cycle-dependent stiffness

degradation parameter. The proposed approach to predict fatigue life is validated with experimental data available in the literature on CFRP reinforced aluminium plates subjected to fatigue loading [30].

7.2 Background on the fatigue behavior of adhesively bonded joints

This section presents some theoretical background and a brief review of past research on the fatigue behavior of CFRP-metal adhesively bonded joints with more focus on the works related to the civil engineering metallic construction sector. There is a growing research interest in the fatigue response of CFRP-metal adhesive joints due to its importance and the limited studies available in the literature. Shenoy et al. [36] investigated the fatigue life of aluminium/epoxy single lap joint. The authors identified three regions in the fatigue life: an initiation period (CI) in which damage starts to accumulate, but a macro-crack has not yet formed, a stable crack growth (SCG) region in which a macro-crack has formed and is growing slowly and a fast crack growth region (FCG), which leads to rapid failure of the joint. The percentage of life spent in each region depends on the fatigue applied load. At low loads, the fatigue life is dominated by crack initiation which leads to long fatigue life, whereas crack growth dominates at high loads. Bocciarelli et al. [37] tested double-sided CFRP bonded on rectangular steel plates subjected to fatigue loading. The results showed that the bond between the steel and the CFRP plate had remarkably superior fatigue life than welded cover plates. Colombi et al. [38] tested double lap joint under fatigue loading with stress ratio, R , equal to 0.1 and 0.4. The results showed that the fatigue resistance of steel/CFRP was not influenced by the stress ratio and the fatigue life of the double shear lap joints is comparable to one of the welded joints (Eurocode 3 design category 40–80). Liu et al. [10] tested steel-CFRP double strap joints with NM and UHM CFRP under fatigue loading. The results revealed that the fatigue loading may produce cumulative damage in the joints, which leads to a reduction of the initial stiffness. Iwashita et al. [39] derived probabilistic S-N curves for steel-FRP single lap joint subjected to fatigue loading. Deng and Lee [40] tested small-scale steel beams bonded with CFRP. The authors noticed that the crack initiated and grown in the adhesive layer in mode 1 before mode 2 and the fatigue was much more influenced by the level of the maximum load than the stress range. The author proposed a fatigue threshold corresponding to 30% of the ultimate failure stress of the adhesive under static loading. Doroudi et al. [41] tested single shear pull-off specimens under

monotonic and cyclic quasi-static loading. The results showed that the cyclic loading has a negligible influence on the bond strength but the stiffness of the load-displacement tends to decrease with the number of cycles, indicating an increase of damage that is highly dependent on the amplitude of the applied loading. Zhang et al. [11] conducted an experimental study on interfacial bond behavior between CFRP sheets and steel plates under fatigue loading using double strap joint specimens. The authors modelled the fatigue life using the S-N approach as well as the fracture mechanics-based approach. Jiao et al. [42] derived S-N curves for double strap joint specimens and showed that the application of a primer resin can reduce the fatigue life significantly. Thay et al. [43] showed that the fatigue life of adhesively bonded joints can be evaluated using the ratio between the principal stress in the adhesive joint over the ultimate principal stress obtained from the tensile test. More recently, based on an experimental database collected from the literature of 36 double strap joint tests, David Jimenez-Vicaria et al. [44] showed that the fatigue data points scatter around the same S-N curve regardless of the structural adhesive used. A design S-N curve was proposed by the authors.

The methods used to model fatigue strength in the adhesive joint vary in complexity, applicability, and degree of empiricism. The standard S-N method is usually adopted although it does not indicate the progression of fatigue damage. Cheuk et al. [45] investigated the fatigue crack initiation and growth in metal-composite bonded double lap joints using experimental and numerical methods. The results revealed that the fatigue in metal-composite double lap joints is mainly driven by mode I due to peel stress since the crack initiation phase and crack growth of short cracks are dominated by mode I. The shear mode becomes dominant as cracks grow longer. Douroudi et al. [46] adopted a damage plasticity model based on mode II bond-slip behavior to capture the constitutive behavior of the bonded interface under quasi-static cyclic and fatigue cyclic loadings. The authors implemented the model numerically using the finite difference method and found that the predictions were conservative as compared to experimental results. The CZM method is the most common way to model the behavior of adhesively bonded joints since it can capture the initiation and propagation of damage for complex mixed-mode stress states. This method is used in the finite element method, where a damage parameter is used to progressively reduce the stiffness, simulating damage growth within the element. Under fatigue

loading, the load history is reduced to a certain number of cycles that are simulated block-wise [35], this method is usually referred to as fatigue cycle jump. However, it requires the fracture mechanics parameters and the relation between the damage variable and the number of cycles which should be determined experimentally. Fatigue stiffness degradation models are successfully adopted to model the fatigue life of composite materials [47], [48]. Except, Zhang et al. [49] who developed models for bonded GFRP double lap joints, and Jiang et al. [50] who adopted them on GFRP/steel double lap joint, fatigue residual stiffness models have not much been used for adhesively bonded joints. However, they may particularly be more advantageous than other methods since they provide information on the evolution of damage and are computationally not expensive. Therefore, more research attention should be given to the development of residual stiffness models for adhesively bonded CFRP-metal joints.

7.3 The proposed approach

The fatigue failure pattern in the metal subjected to tensile cyclic loading is the same for structural detail before and after the reinforcement with patched unidirectional CFRP. Therefore, the S-N curves of the unreinforced structural details can be adopted for the CFRP retrofitted ones, given the reduced stress in the metal is considered [28]. The reduced stress in the metal can be estimated using the finite element method [31] or analytical methods [28], [30], [51], [52]. For example, refs. [28], [33] used simple analytical equations based on the static equilibrium to estimate the stress in the metal of a CFRP reinforced structural detail. For double-sided CFRP reinforcement, the reduced stress in the metal can be estimated using Eq. 1. Note, Eq. 1 is used in this section only for illustration purposes, any other accurate equation that estimates the stress in the metallic adherent can be used.

$$S_{CFRP} = S_{bm} \frac{1}{1 + 2 \frac{E_c \cdot b_c \cdot t_c}{E_m \cdot b_m \cdot t_m}} \quad \text{Eq. 20}$$

where the S_{CFRP} is the stress in the metal for a structural detail reinforced with CFRP, and S_{bm} stands for stress in the base metal which is the stress in the original structural detail without reinforcement when subjected to the same level of loading. The parameters E , b , and t represent respectively the elastic modulus, width, and thickness. The subscripts m denotes metal and c denotes CFRP. Hu et al. [28] proposed to use code-based S-N curves with Eq.1 to estimate the fatigue life for structural details

reinforced with CFRP. So, for a structural detail with a given S-N curve as in Eq. 2, the fatigue life after the retrofit with the CFRP patch can be estimated using Eq. 3.

$$\log N_f = b - a \log S_{bm} \quad \text{Eq. 21}$$

$$\log N_f = b - a \log S_{CFRP} \quad \text{Eq. 22}$$

Thus, for different stress ranges in the finite-life fatigue region, the effect of the composite patch is manifested through a parallel shifting of the S-N curve. In this method, the ratio of increase of fatigue life is constant regardless of the stress level. However, as discussed previously in this chapter, the composite patch undergoes fatigue deterioration that depends strongly on the loading level. And, the ratio of fatigue life extension is not constant but reduces significantly at higher stress [16], [31], [33]. Therefore, it is important to consider the fatigue degradation of the composite patch to possibly provide a more accurate estimation of the fatigue life.

The approach proposed in this study is to modify the equation that estimates the static stress in metal (e.g. Eq. 1) to include the fatigue degradation of the composite patch which is dependent on the number of cycles. A fatigue stiffness degradation model is suitable to represent this behavior [49], [50]. Since the stiffness of the composite patch is going to decrease with the number of cycles [46], the metallic part will be subjected to fatigue loading with increasing stresses. The stress history $S_{CFRP}(N)$ in the metal can be estimated by introducing a cycle-dependent stiffness reduction factor “ $K_{red}(N)$ ” to Eq. 1, as shown in Eq. 4.

$$S_{CFRP}(N) = S_{bm} \frac{1}{1 + 2 \frac{K_{red}(N) E_c \cdot b_c \cdot t_c}{E_s \cdot b_s \cdot t_s}} \quad \text{Eq. 23}$$

Once, the stress history in metal is obtained, the fatigue life can be estimated using a fatigue damage accumulation model such as Miner’s rule, as shown in Eq. 5.

$$D = \sum D_N(S_{CFRP}(N)) \quad \text{Eq. 24}$$

The parameter D is the damage variable that is equal to 1 at failure, D_N is the cycle damage ratio corresponding to the number of cycles N^{th} cycle that depends on the actual stress in the metallic part

$S_{CFRP}(N)$, where D_N is estimated according to Section 6.4. In Figure 7-1, a workflow of the proposed approach is illustrated.

7.3.1 Fatigue stiffness degradation of the adhesive joint

Fatigue stiffness degradation models or also called fatigue residual stiffness models capture the damage evolution and changes in the material properties in function of the number of cycles and stress level. They are widely used to model the fatigue damage evolution for composite materials [47], [48]. However, they were less exploited to model the fatigue life of adhesive joints. Zhang et al. [49] established a linear stiffness degradation model for epoxy adhesive double lap joint composed of pultruded glass fiber reinforced polymer (GFRP) adherents; and a nonlinear model for stepped lap joints. Both of these models were recently adopted by Jiang et al. [50] on GFRP/steel double lap joint. The results showed that the nonlinear model agreed better with the experimental results. In this study, the linear model proposed by Zhang et al. [49] is adopted to model the fatigue residual stiffness of adhesive CFRP/steel double strap joint. This choice is taken for the sake of simplicity and due to the unavailability of sufficient data to calibrate the nonlinear model. The linear model can be calibrated from the S-N curve of the adhesive joint. The model would represent the secant stiffness degradation from the initial loading until failure. Figure 7-4 illustrates the evolution of the debonding length against the number of cycles of double strap joints experimental data used in this study. The experiments were conducted by Zhang et al. [11]. The debonding length which represents the damage of the joint is showing a linear trend, which gives a good indication of the applicability of the linear stiffness degradation model.

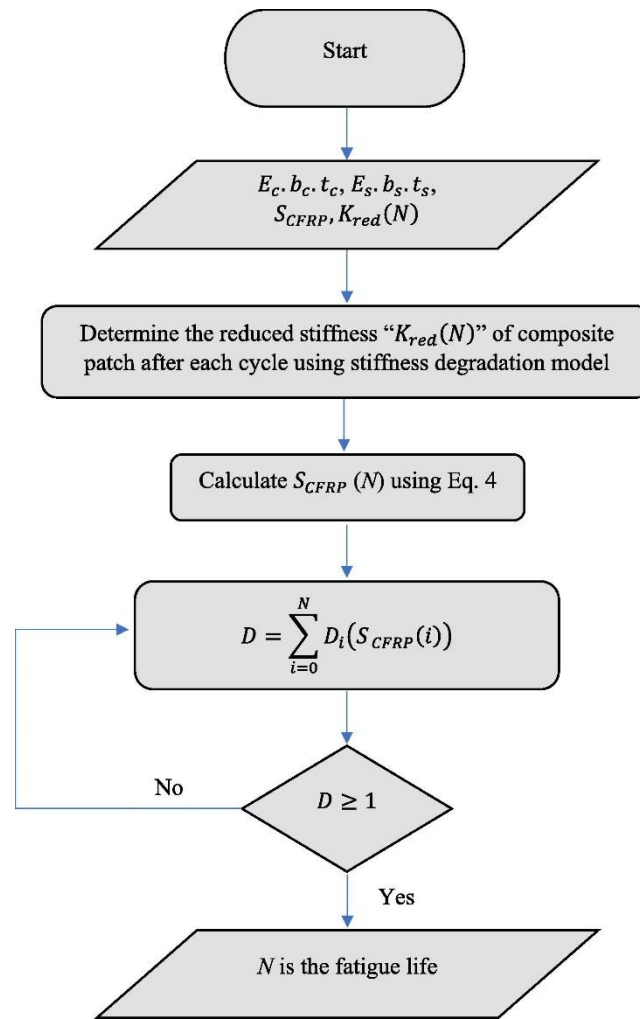


Figure 7-1: Procedure to estimate the fatigue life of CFRP retrofitted structural details.

David Jimenez-Vicaria et al. [44] collected fatigue data from the literature (including the data of Zhang et al. [11] used in this study) and showed that data points for the CFRP/Steel double strap joints with different adhesives scatter around the same S-N curve regardless the surface treatment, CFRP type, adhesive type, and the failure mode. Accordingly, experimental fatigue data of steel/CFRP double strap joint from the literature can be used to calibrate the fatigue residual stiffness model which will be in turn applied for a CFRP-steel reinforcement that might have a composite patch with different properties. However, by precaution, in this study, it is was decided to use the experimental data of Zhang et al. [11] to calibrate the residual stiffness model because the surface treatment of the specimens, the CFRP, adhesive type, the bonding technique are similar to the experimental specimens used for the validation of the approach proposed in this study (see section 6). The geometry of the double strap joint specimens tested by Zhang et al. [11] is shown in Figure 7-2. Unidirectional CFRP sheets of 0.167mm thickness

and elastic modulus of 240 GPa made from filaments produced by Toray Industries Inc [53], were used. The wet layup technique was used to bond the CFRP sheets in both studies using an elastic epoxy adhesive with measured elastic modulus and tensile strain to break around 2.7 GPa and 1.7%, respectively. The experimental data used for the calibration of the residual stiffness model are obtained from 6 CFRP/Steel double strap joint specimens subjected to constant fatigue loading at a frequency of 10Hz. The stress range was applied by increasing the maximum load and keeping the minimum constant. As it was discussed previously, the adhesive joint is strongly dependent on the maximum stress under fatigue loading.

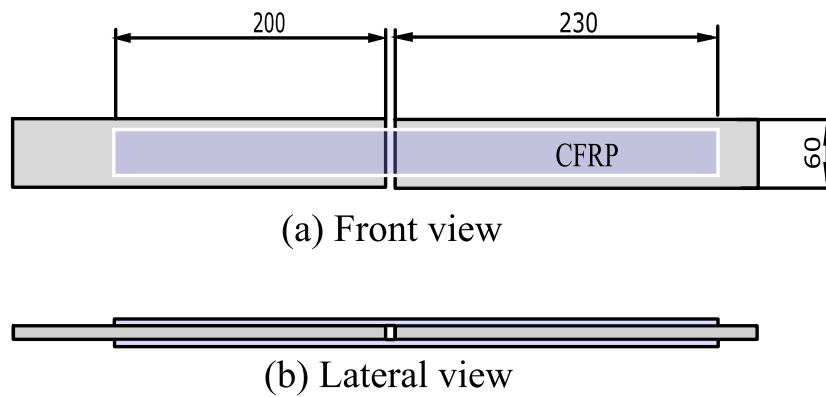


Figure 7-2: Geometry of a double strap joint (dimensions in mm).

The fatigue resistance curve of the adhesive Steel/CFRP double strap joint is presented in Figure 7-3 in the form of load ratio versus the number of cycles (LR-N). The load ratio LR is equal to the applied loading over the ultimate static loading (i.e. F/F_u). The mathematical expression of the LR-N curve is presented in Eq. 6.

$$\log N_f = 8.587 - 6.365 LR \quad \text{Eq. 25}$$

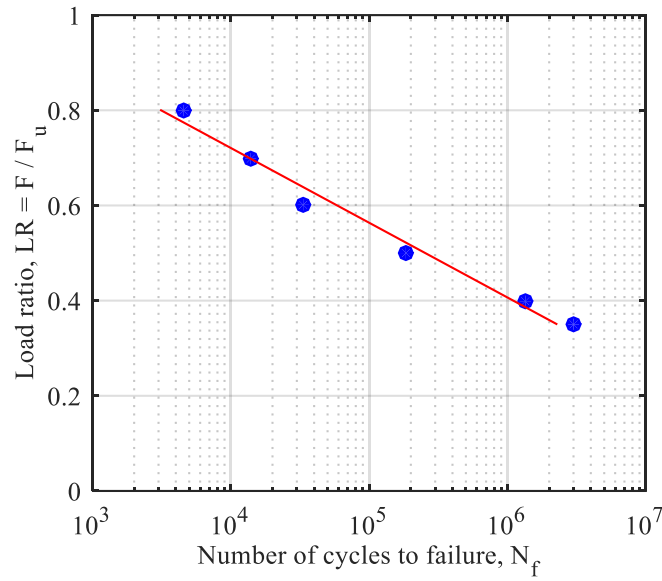


Figure 7-3: S-N curve for double strap joint [11].

Figure 7-4 shows that the fatigue debonding length exhibits a linear relationship with the number of cycles. The debonding length ratio (DLR), equals the debonding length at N^{th} cycle divided by debonding length at ultimate failure, is a parameter that describes the evolution of damage. Therefore, conceptually DLR and $K_{red}(N)$ would represent the same state of the damage. However, since the correlation between DLR (or crack propagation) and the stiffness degradation parameter are not established in the literature, the residual stiffness model is calibrated based on the LR-N curve only. The DLR will be used only for the comparison and validation of the model.

The linear fatigue residual stiffness model proposed by Zhang et al. [49] is presented in Eq. 7.

$$K_{red}(N) = \frac{E(N)}{E(0)} = 1 - k_1(LR)^{k_2}N \quad \text{Eq. 26}$$

where $K_{red}(N)$ is the stiffness degradation parameter of the adhesive joint, $E(0)$ is the initial stiffness, $E(N)$ is the stiffness at N number of cycles, LR is the loading ratio, k_1 and k_2 are the model constants calibrated based on the experimental data. The model is calibrated using nonlinear regression analysis and assuming the $K_{red}(N) = 1$ at failure since complete debonding occurs. The results of the calibration are presented in Eq. 8.

$$K_{red}(N) = 1 - 0.0014 \left(\frac{F}{F_u} \right)^{8.0294} N \quad \text{Eq. 27}$$

Figure 7-5 illustrates the comparison between the stiffness reduction parameter $K_{red}(N)$ and the debonding length damage DLR parameter, where clearly, Eq. 8 either correlates fairly well with the DLR or provides a conservative prediction of the evolution of damage.

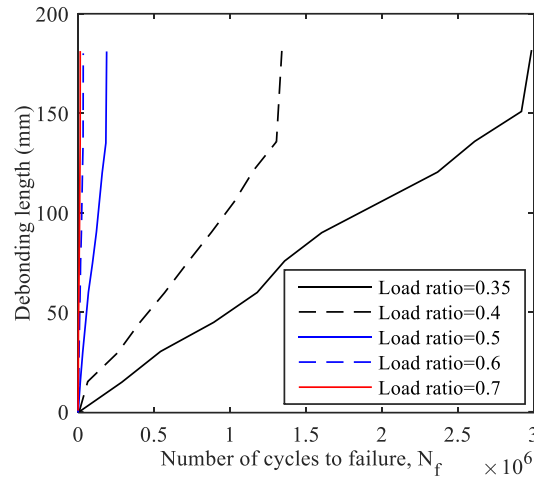


Figure 7-4: Debonding length versus the number of cycles of double strap joint [11].

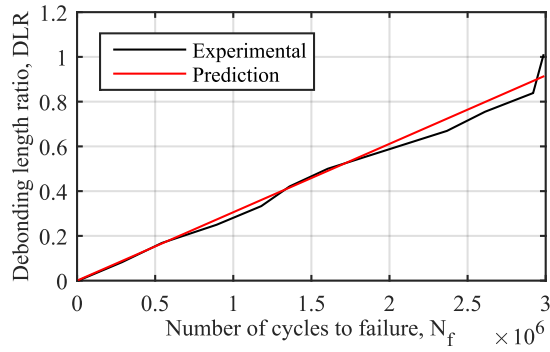
7.3.2 Experimental data of the CFRP reinforced specimens

Very limited fatigue experimental data of non-cracked structural details retrofitted with CFRP are available in the literature. A comprehensive review of the experimental studies on this topic can be found in [16]. Wang et al. [30] tested aluminium alloy plates with a single fastener hole in the middle patched with CFRP subjected to fatigue loading. To the best knowledge of the author, this is the largest set of fatigue data on non-cracked structural details retrofitted with CFRP available in the literature. The specimens were tested under different stress ranges in the finite life region which makes them suitable to validate the approach proposed in this study. Details related to tests of Wang et al.[30] are the following:

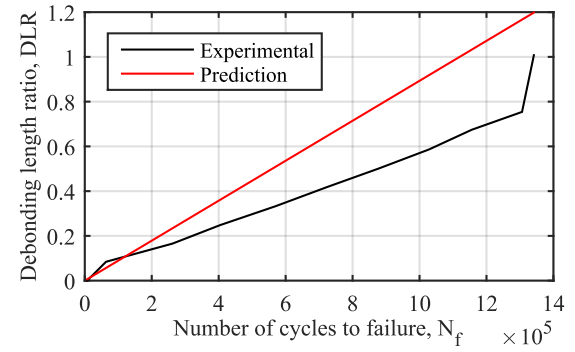
- The dimensions of the aluminium specimens are $240mm \times 30mm \times 2mm$ with a fastener hole of diameter 5mm in the middle, as shown in Figure 7-6 (a). The CFRP dimensions are $140mm \times 30mm \times 0.167mm$ with a central hole of diameter equal to 5mm, as shown in Figure 7-6 (b).

- The experimental data for specimens with double-sided configuration are considered in this study, which consists of 23 reference specimens without reinforcement (Iv-P0-S0) and 20 specimens reinforced with CFRP on the two sides of the plate (Iv-P0-SD1L).
- The specimens are made of grade 2024-T3 aluminium alloy with average yield stress and ultimate stress equal to 307MPa and 405MPa, respectively.
- The unidirectional CFRP sheets UT70-30 of 0.167mm thickness commercialized Toray Industries Inc are used for reinforcement. The CFRP has an elastic modulus of 252 GPa and elongation to break of 1.76%.
- The wet-layup technique was adopted to impregnate the CFRP sheets using an elastic epoxy adhesive (SIKADURE 330). The elastic modulus and tensile strain to break are between 3.5-4.5 GPa and around 1% [54], [55], respectively.
- The fatigue loading was applied under the stress ranges between 80-300 MPa until complete failure of the aluminium plate with a stress ratio $R = 0.1$, with loading frequency of 8 Hz.
- Figure 7-7 shows the fatigue experimental data of unreinforced and reinforced specimens as well as the prediction of the fatigue life without considering the fatigue degradation of the composite patch which estimates reasonably well the fatigue life of reinforced specimens at low-stress levels but over-estimates it significantly at higher levels of stress. The fatigue data points of the reinforced specimens at low-stress levels (i.e. $\Delta\sigma \leq 125 \text{ MPa}$) are lower than the mean S-N curve. In here, it is assumed mainly due to the inherent scatter of fatigue data which is usually higher at low-stress levels. This can be considered using probabilistic methods, however this is beyond the scope of this study. Referring to Figure 7-8, the extension of fatigue life for reinforced specimens tends to decrease for increasing stress levels, where the fatigue life extension ratio (i.e. $Ratio = N_f(reinforced)/N_f(bare)$) is decreasing for increasing stress level. Note that the $N_f(retrofitted)$ and $N_f(bare)$ values, used to calculate the $Ratio$, were obtained using linear fitting of the data. This reduction of fatigue life is mainly due to the fatigue degradation of the composite patch at a higher stress level where the CFRP reinforcement becomes less effective. This observation is in line with other experimental studies discussed previously in this study. Therefore,

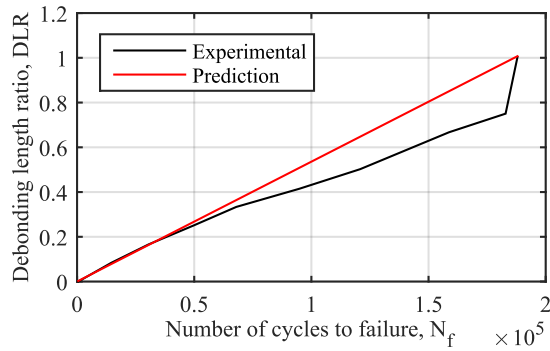
the development of a procedure that takes into account the fatigue degradation of the composite patch such as the one proposed in this study is of paramount interest to provide more accurate predictions of the fatigue life for CFRP reinforced structural details.



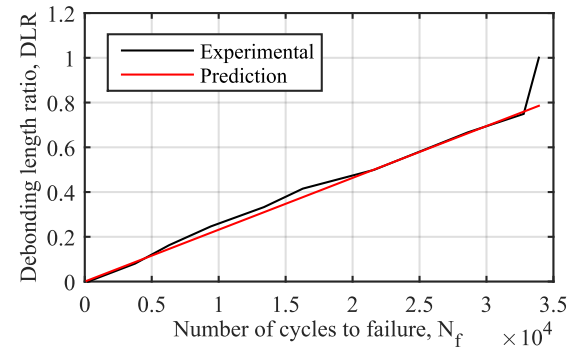
(a) Load ratio LR=0.35



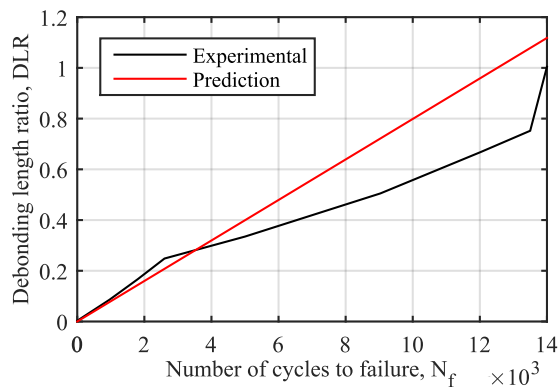
(b) Load ratio LR=0.4



(c) Load ratio LR=0.5



(d) Load ratio LR=0.6



(e) Load ratio LR=0.7

Figure 7-5: Comparison between Experimental damage parameter “DLR” (i.e. Debonding length/Debonding length at failure) [11] and the residual stiffness damage parameter $K_{red}(N)$ versus Number of cycles to failure.

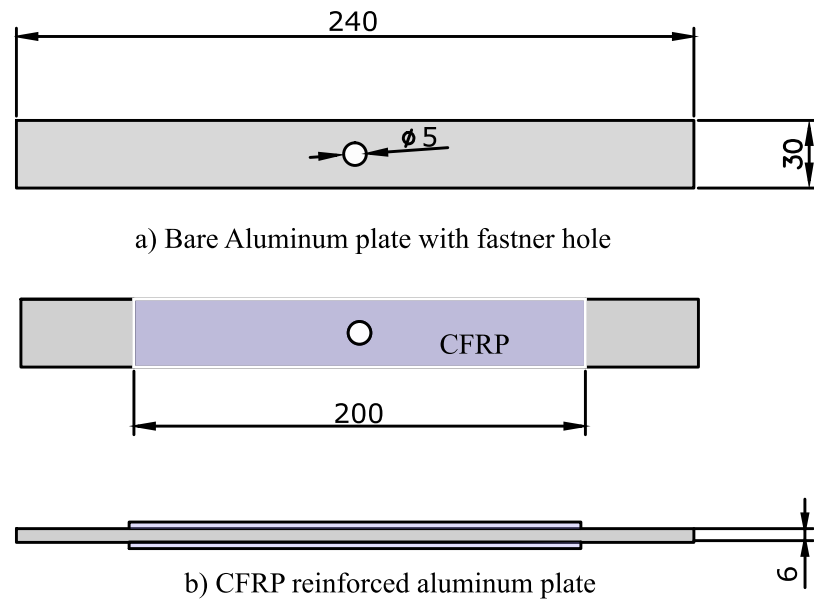


Figure 7-6: Specimens with and without CFRP (dimensions in mm).

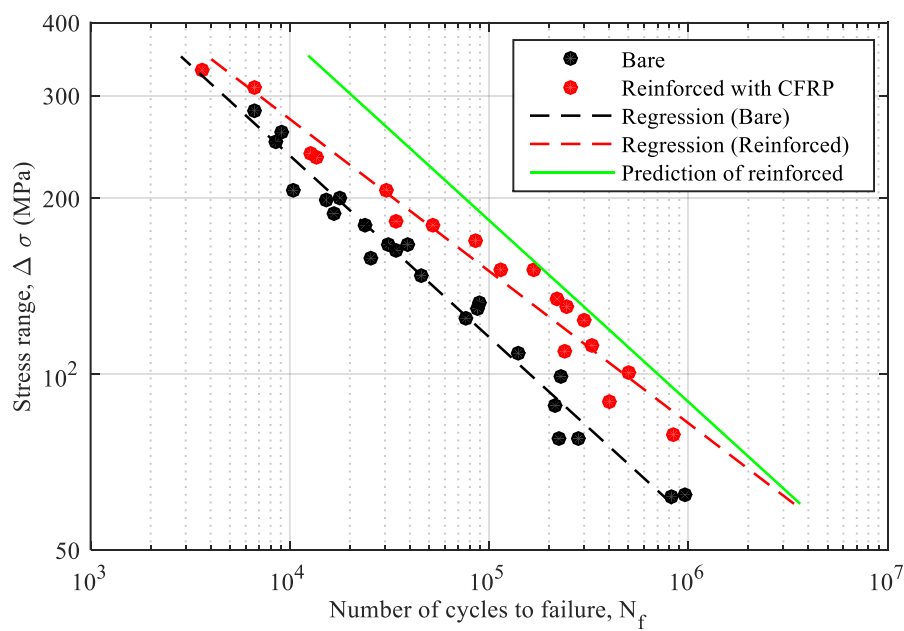


Figure 7-7: Prediction of reinforced specimens with CFRP without considering the fatigue degradation of the composite patch.

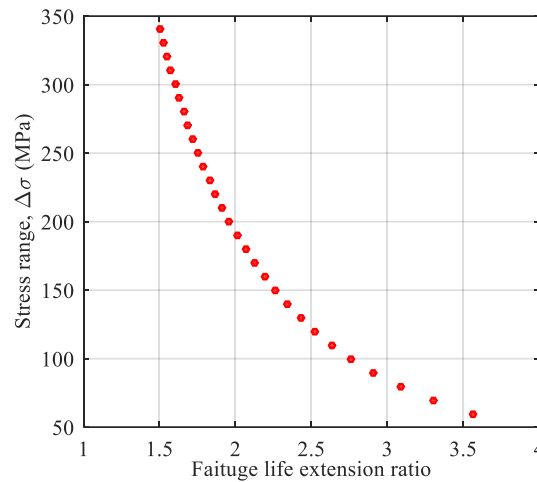


Figure 7-8: Ratio of increase of fatigue life versus the stress range.

7.3.3 Validation of the proposed approach

In this section, the proposed approach presented in section 3 is applied for CFRP reinforced specimens presented in section 5 for validation. To estimate the fatigue life of CFRP reinforced specimens according to the approach proposed in section 2, these steps need to be followed:

- 1- Determine the S-N curve for bare metallic specimens without CFRP reinforcement (see section 6.1).
- 2- Evaluate the stress in the adhesive layer in CFRP reinforced specimens, calculate the load ratio which will be in turn used to estimate the $K_{red}(N)$ for each stress range using Eq. 8. (see section 6.2).
- 3- Determine the stress history in the metallic part $S_{CFRP}(N)$ using Eq. 4 for a large number of cycles (e.g. $N = 1$ to 5×10^6). (see section 7.6.3)
- 4- Once, $S_{CFRP}(N)$ is obtained for a large number of cycles, a cumulative damage rule can be applied to estimate the fatigue life. (see section 7.6.4)

7.3.3.1 S-N curve for unreinforced specimens

In the finite life region, there is an underlying linear relationship between the stress and the number of cycles to failure in a double logarithmic scale that can be modelled using the Basquin's [56] relation as given in Eq. 9. To further simplify the analysis, Eq. 9 is transformed into Eq. 10, where x is $\log N_f$ and y is $\log \Delta\sigma$.

$$\log N_f = b - a \log \Delta\sigma \quad \text{Eq. 28}$$

$$x = b - a y \quad \text{Eq. 29}$$

where a and b are constants that are usually obtained by fitting experimental data using the maximum likelihood method, $\hat{a}$ can be obtained using Eq. 11 and $\hat{b}$ using Eq. 12. The caret sign “^” is used to indicate that they are estimates.

$$\hat{a} = \frac{\sum_{i=1}^n (x_i - \bar{x})(y_i - \bar{y})}{\sum_{i=1}^n (y_i - \bar{y})^2} \quad \text{Eq. 30}$$

$$\hat{b} = \bar{x} - \hat{a}\bar{y} \quad \text{Eq. 31}$$

For its simplicity, this model is adopted in most fatigue standards such as ISO12107 [57], ASTM E739 [58], and Eurocode 3 part 9 [23]. However, this model is limited only to the linear region of the fatigue life. To model the fatigue life full fatigue life regimes other models should be adopted [59]. Kohout and Věchet (K-V) [60] proposed a model that estimates the fatigue life ranging from the ultimate tensile strength to the high-cycle fatigue region. The mathematical expression of the K-V model is presented in Eq.13. Note the K-V model can be applied using different damage parameters such as strain parameter, Smith-Watson-Topper (SWT) parameter, Walker-like strain parameter, energy-based parameter [61].

$$\sigma(N) = \sigma_{\infty} \left(\frac{N_f + B}{N_f + C} \right)^b \quad \text{Eq. 32}$$

where σ_{∞} is the fatigue endurance limit, B is the number of cycles corresponding to the intersection of the tangent line of the finite life region and the horizontal asymptote at the ultimate static strength (σ_u), and C is the number of cycles corresponding to the intersection of the tangent line in the finite life region and the horizontal asymptote at the fatigue endurance limit.

In this study, Basquin’s model and the K-V model have been adopted to model fatigue life. Basquin’s model is used to estimate the number of cycles to failure when applying Miner’s damage rule. Whereas, the K-V model is used when applying a sequential fatigue damage model that requires information on the static strength and the low cycle fatigue regime. Figure 7-9 shows the fitting curves for the bare aluminium specimens using the Basquin and the K-V models. It can be seen both models correlate fairly well with the experimental data in the finite life region. But additionally, the K-V model predicts the

static strength and the low cycle fatigue life. Note since the static strength (i.e. $\sigma_u = 421 \text{ MPa}$) was not reported in the experimental data, it was obtained from a finite element nonlinear simulation in ABAQUS®. The true stress-strains of the material were utilized. Details of the numerical modelling are not reported herein for brevity. The prediction of fatigue life at lower stress levels are not useful for the sequential damage model since aluminium alloys do not exhibit a fatigue limit, σ_∞ is assumed to equal the unity. The fitting results of the Basquin and K-V models are presented in Eq. 14 and Eq. 15, respectively.

$$\log N_f = 11.66 - 3.23 \log \Delta \sigma \quad \text{Eq. 33}$$

$$\sigma(N) = \left(\frac{N_f + 1526.91}{N_f + 4.59 \times 10^{11}} \right)^{-0.3095} \quad \text{Eq. 34}$$

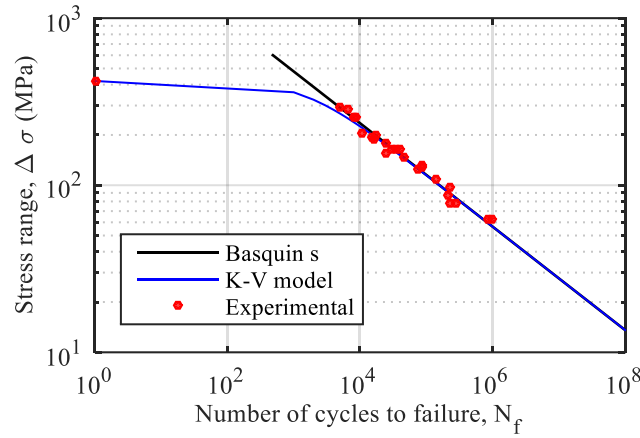


Figure 7-9: Fatigue life curves for bare aluminium plates.

7.3.3.2 Stress in the Adhesive layer

For a structural detail geometry as shown in Figure 7-6, the shear stress inside the adhesive can be estimated using Eq. 16 as provided in Wang et al. [51].

$$\tau_a = \frac{\lambda \Delta \sigma \sinh(\lambda c)}{[2 + E_s A_s (E_c A_c)^{-1}] \cosh(\lambda l_0) ds} \quad \text{Eq. 35}$$

where τ_a is the shear stress in the adhesive, G_a is the shear modulus of the adhesive, x is the distance from the reference point taken in the centre of the specimen, ds is a unit section taken equal to unity, l_0 is the distance between the centre and the edge of the composite patch, and t_a is the thickness of the adhesive layer. The elastic shear stress distribution parameter λ can be estimated using Eq. 14.

$$\lambda^2 = \frac{G_a ds}{t_a} \left(\frac{1}{E_c A_c} + \frac{2}{E_s A_s} \right) \quad \text{Eq. 36}$$

The thickness of the adhesive may not remain regular along with the specimen even at controlled laboratory setup conditions [30] and the measurement of the thickness is a challenging task. For wet layup impregnation, it was experimentally measured for a large number of specimens between 0.3-0.5 mm [62], [63]. Therefore, the thickness is assumed to be 0.4mm in this study. For SIKADURE330, the ultimate shear stress and the shear modulus are taken equal to $\tau_u = 29.5 \text{ MPa}$ and $G_a = 1.72 \text{ GPa}$ based on experimental results reported in [64]. The maximum shear stress of the adhesive layer is located at the ends of the composite patch [65] where $x = l_0$. Note the shear stress near the hole where there is stress concentration can be higher than the shear stress at the ends of the composite patch, however, this region can be assumed unbonded. This assumption is widely adopted in modelling CFRP repaired cracked structural details [66]. Table 1 presents the maximum shear stress in the adhesive as well as the stress ratio which is needed to be implemented in Eq. 7 to evaluate the residual stiffness $K_{red}(N)$. Several failure hypotheses have been considered in the literature for the analysis of adhesive joints [67] [68]. For simplicity, in this study, the maximum shear stress is taken as the failure criterion and the load ratio (LR) is calculated based on the ultimate shear stress of the adhesive (i.e. $LR = \tau_a / \tau_{ultimate}$).

7.3.3.3 Evaluation of the stress history in reinforced specimens

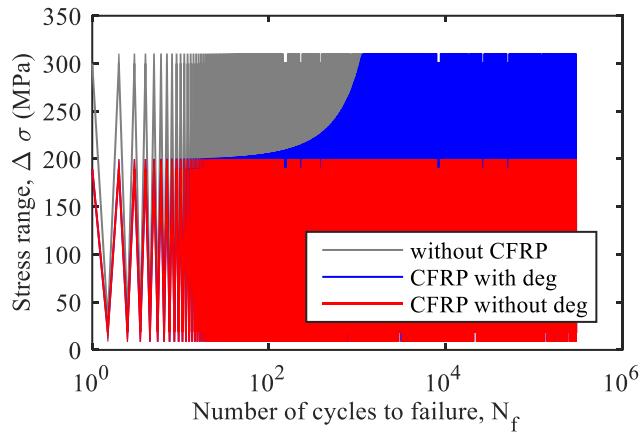
Once, the load ratio in the adhesive layer is determined (see Table 1), the cycle-dependent residual stiffness $K_{red}(N)$ can be estimated for each stress range using Eq. 8. This must be done for a very large number of cycles (e.g. $N = 1 \text{ to } 5 \times 10^6$). Then, stress history $S_{CFRP}(N)$ can be obtained using Eq. 4. Figure 7-10 presents some examples of cyclic stress histories, which were obtained using Eq. 4, for specimens subjected to different constant loading stress ranges. The figure presents the stress in specimens without CFRP reinforcement, with CFRP but without considering the fatigue degradation of the composite patch, and with CFRP and considering the fatigue degradation of the composite patch. Only, stress histories of a reduced number of cycles are presented to save computation time.

The stress for specimens with CFRP considering the stiffness degradation of the composite patch starts at the same stress level of specimens with CFRP but without considering the fatigue degradation, then the stress starts to increase due to the fatigue degradation of the composite patch until complete

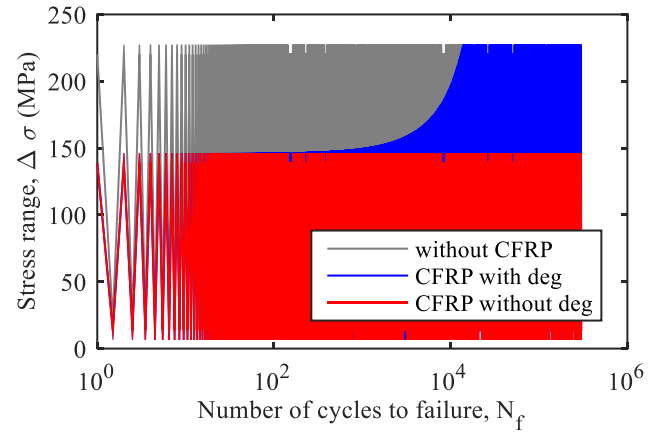
debonding where the stress becomes equal to the specimens without CFRP. The rate of increase of stress depends on the load ratio (LR) which is directly related to the applied stress range. At high-stress ranges ($\Delta\sigma > 200MPa$), the degradation starts very early ($N_f < 10^3$) as in the case of Figure 7-10 (a, b). The CFRP reinforcement is effective only in a limited number of cycles. Therefore, the CFRP reinforcement is expected to have a negligible effect on the fatigue life of the specimens. At medium stress ranges ($150MPa \leq \Delta\sigma \leq 200MPa$), the composite patch starts to degrade around $N_f = 10^4$. In this case, the composite patch has an effect on the fatigue life, and it is important to consider it. At low-stress ranges ($\Delta\sigma \leq 100MPa$), the composite patch does not exhibit considerable degradation. The CFRP reinforcement is very effective in reducing stress for a very long life. Thus, the fatigue life is expected to be considerably extended.

Table 7-1: Maximum shear stress and stress ratio in the adhesive layer.

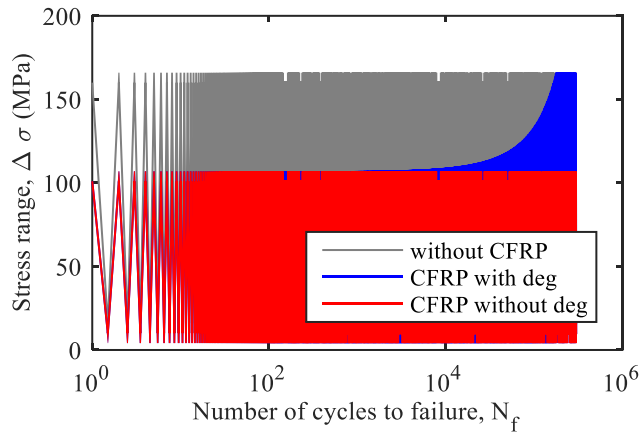
$\Delta\sigma$ (MPa)	t_a (mm)	G_a (MPa)	τ_{max} (MPa)	τ_u (MPa)	LR
80	0.4	1750	7.45	29.5	0.25
90	0.4	1750	8.38	29.5	0.28
100	0.4	1750	9.31	29.5	0.32
110	0.4	1750	10.24	29.5	0.35
120	0.4	1750	11.17	29.5	0.38
130	0.4	1750	12.10	29.5	0.41
140	0.4	1750	13.0	29.5	0.44
150	0.4	1750	13.96	29.5	0.47
160	0.4	1750	14.89	29.5	0.50
170	0.4	1750	15.82	29.5	0.54
180	0.4	1750	16.75	29.5	0.57
190	0.4	1750	17.68	29.5	0.60
200	0.4	1750	18.61	29.5	0.63
210	0.4	1750	19.54	29.5	0.66
220	0.4	1750	20.48	29.5	0.70
230	0.4	1750	21.41	29.5	0.73
240	0.4	1750	22.34	29.5	0.76
250	0.4	1750	23.27	29.5	0.79
260	0.4	1750	24.20	29.5	0.82
270	0.4	1750	25.13	29.5	0.85
280	0.4	1750	26.06	29.5	0.88
290	0.4	1750	27.00	29.5	0.91
300	0.4	1750	27.92	29.5	0.95



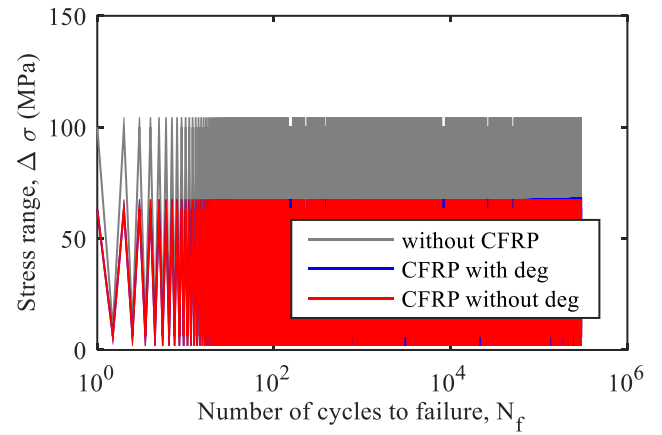
(a) $\Delta\sigma = 300\text{MPa}$, $LR = 0.95$



(b) $\Delta\sigma = 220\text{MPa}$ and $LR = 0.7$



(c) $\Delta\sigma = 160\text{MPa}$ and $LR = 0.5$



(d) $\Delta\sigma = 100\text{MPa}$ and $LR = 0.3$

Figure 7-10: Stress histories in metal with and without considering the stiffness degradation of the composite patch.

7.3.3.4 Fatigue damage accumulation models

Palmgren first presented the concept of the linear accumulation of damage. Miner [69] presented its mathematical form assuming that the total work to cause failure in a specimen is constant regardless of the amplitude of the loading. Thus, the total work (W_T) for a sample subjected to a spectrum of loading of i block is:

$$\sum w_i = W_T \quad \text{Eq. 37}$$

Assuming that the number of cycles is proportional to the number of blocks gives

$$\frac{w_i}{W_T} = \frac{n_i}{N_f} \quad \text{Eq. 38}$$

where n_i is the number of cycles at a given stress range $\Delta\sigma_i$, and N_f is the number of cycles to failure that can be obtained from S-N curves. Therefore, from Eq. 18 and Eq. 19, Miner's linear accumulation damage model is developed as presented in Eq. 20.

$$D = \sum r_i = \sum \frac{n_i}{N_f} \quad \text{Eq. 39}$$

The parameter r_i is the cycle damage ratio corresponding to the i^{th} load level, For its simplicity, this method is commonly used to predict variable amplitude fatigue life [70]. It assumes a linear fatigue damage accumulation with no load history effect. Failure is deemed to occur when the sum of $\sum n_i/N_f$ is equal to 1. The main disadvantages of this method are the load-level independence, load sequence independence and lack of load interaction accountability [71]. The model leads to optimistic results with decreasing loading conditions and pessimistic results with increasing loading conditions [72]. Moreover, the rule also neglects damage raised by stresses below the fatigue limit [70].

Many different damage accumulation models for metals and their alloys have been published in the literature [70], [73]–[76]. They can be classified into two categories: (1) linear damage cumulative theories and (2) nonlinear damage cumulative theories [77]. Macro and Starkey [78] firstly introduced a nonlinear load-dependent damage rule. However, nonlinear models usually require more parameters that need to be calibrated from experimental data. Mesmacque et al. [72] presented a sequential model (see Eq. 21). The model overcomes the limitation of Miner's damage rule and does not require too many properties. This model is based on the damage stress concept and is connected cycle by cycle with Wöhler's curve.

$$D_i = \frac{\sigma_{edi} - \sigma_i}{\sigma_u - \sigma_i} \quad \text{Eq. 40}$$

where D_i is the damage parameter, σ_{edi} is the damage stress that needs to be evaluated after each cycle, σ_u is the ultimate stress and σ_i the applied stress.

Pastor et al. [31] found that the sequential model provided the best correlation with experimental fatigue data of aluminium specimens patched with CFRP. Therefore, it is adopted in this study to estimate the fatigue life of CFRP reinforced specimens. Miner's rule is also provided for the sake of comparison.

7.3.3.5 Results and Validation

Figure 7-11 presents the estimated fatigue life for CFRP reinforced specimens using Miner's damage rule. The fatigue data of the bare and CFRP reinforced aluminium plates are presented. It can be seen that the approach proposed in this study could capture the trends of the fatigue data for CFRP reinforced specimens, especially at high stresses, while the existing methods (without fatigue degradation) cannot capture this behaviour.

The fatigue curve derived using the proposed approach and the one without fatigue degradation match at low stress levels, this was expected as discussed previously which is mainly due to the insignificant stiffness degradation of the composite patch at low stress levels as shown in Figure 7-10. However, the fatigue experimental data points of CFRP reinforced specimens become closer to the S-N curve of bare metallic specimens without reinforcement as the stress level increases. This is mainly due to the fatigue degradation of the composite patch. As it can be seen in Figure 7-11 and Figure 7-12, the prediction without fatigue degradation does not capture this behavior and overestimate the fatigue life significantly. Whereas the proposed approach captures reasonably well the fatigue life for high stress levels.

Figure 7-12 presents the estimated fatigue life for CFRP reinforced specimens using the sequential damage model. Similar to Miner's rule, the sequential model could capture the trends of the fatigue data for CFRP reinforced specimens. However, at high stresses, the model presents more conservative results. This is mainly due to the deviation of the K-V model in the transition between medium to low cycle fatigue, which was calibrated by interpolation between the fatigue data in the medium cycle regime and the static strength. Therefore, additional fatigue data of bare aluminium plates should re-adjust the model to fit better the data otherwise other fatigue models that can capture the full fatigue life can be used such as the logistic model [79]. However, generally, the results are fairly acceptable

compared to existing methods (without fatigue degradation) and further validation with new experimental data are recommended for variable amplitude loading.

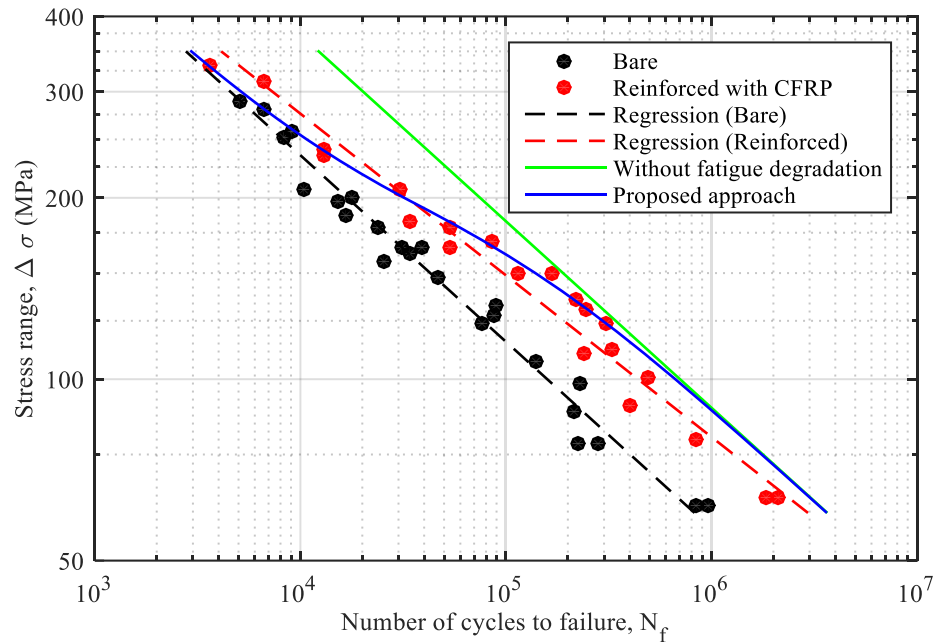


Figure 7-11: Prediction based on Miner's rule.

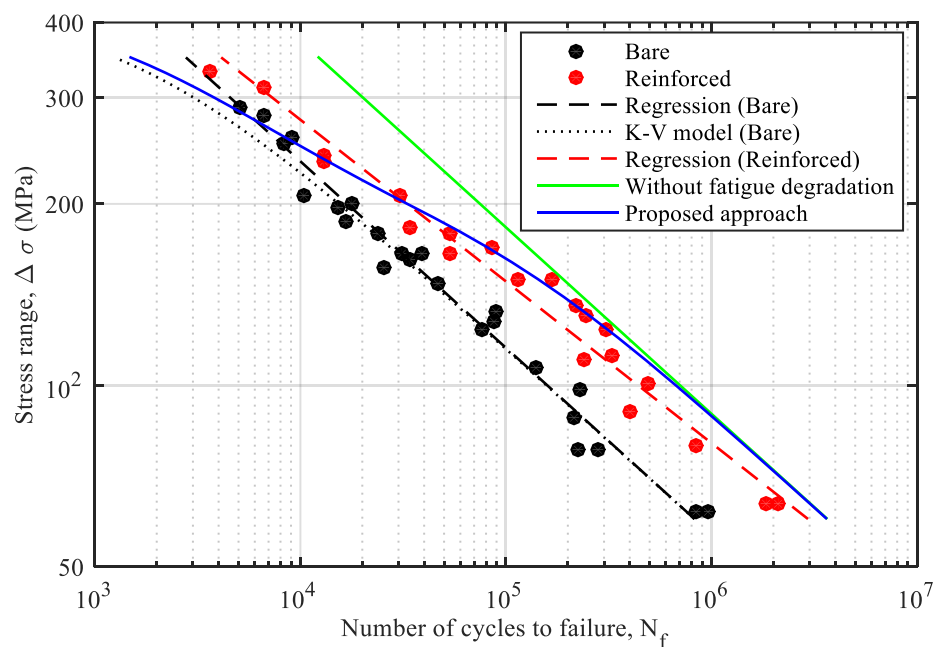


Figure 7-12: Prediction based on the sequential model.

7.4 Concluding remarks

This chapter focuses on the prediction of fatigue life of metallic structural details reinforced with CFRP. Most studies available in the literature do not consider the fatigue degradation of the composite patch when the structural detail is subjected to fatigue loading. To the best knowledge of the authors, except for some computationally expensive numerical procedures such as the fatigue cohesive zone modelling cycle jump, there are no simplified analytical methods proposed in the literature to estimate the fatigue life of CFRP patched structural details which considers the fatigue degradation of the composite patch. In this study, a simplified approach is proposed based on a fatigue phenomenological residual stiffness model of the composite patch and fatigue damage accumulation rules of the metallic structural detail. The approach is fairly simple compared to existing numerical methods and has been validated with an experimental fatigue dataset collected from the literature on aluminium plates featuring a hole in the middle and patched on both sides with CFRP sheets.

At a low-stress level, the application of CFRP on metallic structural details increases the fatigue life by a constant ratio that is mainly dependent on the stiffness of the composite patch. However, the extension ratio tends to decrease as the stress level increases. This is mainly due to the influence of the fatigue loading on the composite patch. At higher stress levels, the composite patch experiences fatigue degradation. Using the proposed approach, the reduction of the extension of fatigue life at higher stress levels could be fairly well predicted. The results obtained using the proposed approach follow well the trend of the experimental data, whereas the main method used in literature, which does not consider the fatigue degradation of the composite patch, overestimates the fatigue life significantly.

This research was intended to conceptually present and validate the approach. Therefore, a simplified fatigue stiffness degradation model, composite patch failure criterion, and fatigue cumulative damage model were adopted. Future research studies should address the refinement of the approach using nonlinear stiffness degradation models and other failure criteria of the composite. More advanced fatigue damage accumulation models considering variable fatigue loading should also be considered. Note that for more complex non-symmetric geometries such as one-sided patches that may induce bending, the method proposed in this study maybe not very appropriate. However, the same concept

maybe applied but needs to be combined with the finite element method to capture the demands in the adhesive layer and the metallic components. Then, a step-wise analysis, similar to the cycle jump used in fatigue cohesive zone modelling, can be used.

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Chapter 8: Fatigue crack growth modelling for cracked small scale structural details repaired with CFRP

8 Fatigue crack growth modelling for cracked small-scale structural details repaired with CFRP

Abstract

This chapter presents an analytical model to predict the mode I fatigue crack growth of steel Central Cracked Tensile (CCT) specimens repaired with Carbon Fiber Reinforced Polymer (CFRP) bonded on the two sides of the plate. A review of the behaviors observed in past experimental tests is provided. The CFRP-steel stiffness ratio, as well as the level of the initial crack length and the location of the CFRP patch, influence the performance of the CFRP crack repair. A total of 69 experimental and 7 numerical results of CFRP repaired specimens were collected from the literature and a database is created. Seventy percent of the data (Experimental only) were used for the development of the model and thirty percent for verification. The fatigue crack propagation of each pair of unrepaired and repaired experimental specimens was calibrated for the development of the model. The effect of the CFRP on the fatigue crack propagation is represented by a SIF reduction factor, β . An analytical law based on nonlinear regression analysis is proposed to evaluate β which depends on the CFRP-steel ratio, level of damage of the steel plate, and patch location. The model is fairly validated with experimental and numerical results. A parametric study is conducted for further verification, the model shows reasonable trends.

8.1 Introduction

The maintenance of metallic bridges around the world is a critical issue. Statistical survey on metallic bridge failures showed that fatigue-related damages represent a major percentage [1]. Several experimental [2]–[4] and numerical studies [5]–[11] were conducted in the understanding of fatigue crack propagation of metallic components and in developing repair methods to delay or arrest the fatigue crack propagation. The retardation of the crack growth in a cracked structural details can be achieved by several repair techniques, where some of the most effective ones are: stop-holes [12], [13] and the application of CFRP patch near the crack tip. When there is a possibility, the combination of the two methods seems to be a judicious engineering solution however it received little research attention in the context of bridge engineering [14]. Owing to the superior properties of the material, fatigue crack repair techniques using Fiber Reinforced Polymer (FRP) are well established in aerospace and

aeronautical engineering for lightweight structures [15], [16]. In the last two decades, lot of research attention was given for the application of CFRP patch to extend the fatigue life in metallic bridges [4], [17]–[20]. In order to implement this technique in practice, simple and accurate design procedures are required to be developed and implemented in design guidelines.

Several experimental and numerical researches were conducted on CCT specimens representing a fatigue crack emanating from a connection hole [21]–[29]. This pathology was observed in full-scale bridge girder fatigue testing [4]. Experimental evidence showed very promising results of CFRP crack repair, since in most cases the fatigue life could be extended considerably, especially when the composite plate is prestressed [29]–[33] or a CFRP with high elastic modulus (i.e. $E > 400 \text{ GPa}$) was used [34], [35]. The CFRP patch shares the average stress with the steel element, thus reducing the stress intensity factor (SIF). The bridging effect provided by the CFRP near the crack tip limits crack-opening [36]. The performance of the fatigue crack repair by bonded CFRP depends on several factors, such as: i) the mechanical and geometrical properties of the composite material, ii) the location of the patch, and iii) the initial degree of damage of the steel plate. To predict the crack propagation life of CFRP repaired cracks, fatigue crack growth (FCG) models based on Paris law are usually adopted. However, challenges remain in the estimation of the reduction in SIF due to the application of CFRP. Since several parameters influence the crack growth behavior of a CFRP repair, the development of an exact analytical solution is almost impractical [37].

Simplified models with empirical correction factors are usually adopted to estimate the reduction in SIF for repaired specimens with CFRP. For mode I crack propagation, several studies focused on the estimation of the SIF in the context of bridge and steel construction engineering. Wang and Nussbaumer [36] proposed to include the instantaneous stiffness contribution of the CFRP according to the evolution of crack length to estimate the SIF, then a correction factor was proposed to match experimental data. Liu et al. [38] proposed a procedure to estimate the stiffness contribution of each layer of CFRP sheets and applied the FCG model proposed by Newman [39] that includes the crack closure effect to predict the fatigue crack propagation of a CFRP repair for double-sided CFRP configuration. Wu et al. [40] proposed simplified modification factors that take into account the material properties

and the geometry of the CFRP to estimate the reduced SIF for CCT specimens fully covered by CFRP. Yu et al. [41] estimated the reduction in the average stress due to the stress sharing effect of the CFRP using elastic stress analysis theory, and based on the finite element method and elastic fracture mechanics principles, a geometrical correction factor was proposed to be added in the calculation of the SIF. Wang et al. [37] provided an analytical expression based on theoretical stress analysis and correction factors to estimate the SIF for double edge crack specimens validated with numerical results. Colombi et al. [42] proposed an analytical FCG model developed based on numerical analysis and experimental results for Single Edge Notched Tension (SENT) specimens. Wang et al. [43] used the finite element method to estimate the SIF of small scale specimens, which was modified to include the crack closure effect as provided by Elber [44]. Liu et al. [45] performed a large number of finite element simulations and provided multiple empirical formulations to estimate the SIF correction coefficient. Do and Lenwari [46] also proposed a correction factor developed by regression via Genetic programming analyses using the SIF values obtained from the finite element simulation. While most studies focused on the mode I fatigue crack propagation, some others studied the mixed mode crack propagation [7], [47]–[49]. Conceptually the effect of bonded unidirectional CFRP (with fibers parallel to the loading) on mode I crack propagation could be extended for mixed mode conditions. Since the SIF mode I component (K_I) is usually dominant in mixed mode FCG and the efficiency of the CFRP is not pronounced for specimens having the crack driving force mainly dominated by mode II SIF [49]. The reduction factor developed for representing the effect of CFRP in mode I only, can be applied on (K_I) to estimate the mixed mode effective SIF when using the maximum tangential stress criterion (MTS) [50], [51], or other methods [52], [53].

Most of the aforementioned studies that focused on predicting mode I FCG for cracked specimens repaired with CFRP were based on limited experimental data and did not comprehensively consider the combination of the most relevant parameters that can influence the fatigue crack growth of a CFRP repair. The more recent models are based on numerical finite element results where interface damage and deterioration of the bond during fatigue loading are not considered. In the last decade, many experimental tests have been published in the literature [21]–[28] and the state of knowledge was considerably improved. Therefore, there is a need to develop a prediction

model to estimate the SIF of crack repair with CFRP patches that includes the most relevant influencing parameters observed recently in physical tests.

This study focuses on the prediction of the mode I SIF for steel CCT specimens repaired with bonded CFRP on the two sides of the plate. A review of the experimental observations reported in the literature is first provided to have a clear view of the fatigue crack propagation response of CCT specimens repaired with CFRP and identify the parameters that influence their performance. Experimental results collected from the literature are presented. The fatigue crack propagation model was calibrated for unrepaired and repaired specimens. The influence of the CFRP repair was evaluated by means of SIF reduction factor “ β ”. Nonlinear regression analysis was conducted to provide an analytical equation that can estimate β based on the most influencing parameters observed in the experimental results. The regression variables include the CFRP- steel stiffness ratio, the initial crack length ratio and a parameter that takes into account the location of the CFRP patch with respect to the notch tip. The predictions of the model are compared with 16 experimental and 7 numerical results that were not used for its calibration. Finally, a parametric study is conducted to assess the influence of each parameter of the model.

8.2 Experimental studies on fatigue CFRP repairs

8.2.1 Literature review on experimental results

Past experimental tests on cracked metallic small-scale specimens repaired with bonded CFRP patches subjected to fatigue loading showed promising results. Runouts and crack propagation arrests could be achieved with high modulus (HM) CFRP ($E > 400\text{GPa}$) [34], [35]. Good results also could be achieved when using normal modulus (NM) CFRP (i.e. $E < 300\text{GPa}$). For instance, Bassetti et al. [4] showed that the propagation fatigue life could be extended by at least a factor of three when using NM CFRP with no prestress. Täljsten et al. [28] also conducted similar tests and found that fatigue life could be improved by a factor ranging between 2.44 and 3.75. The performance of a CFRP repaired crack depends on several parameters related to the patch dimensions and location, the stiffness ratio, the level of damage of the specimen, the CFRP system, the properties of the adhesive layer, and so forth. The stiffness of the composite patch is, unquestionably, the most influencing parameter that contributes

to extending fatigue life ([21], [26], [28], [29]). Experimental tests [21], [25], [26], [54] which focused on the effect of the patch geometrical configuration showed that single-sided configuration introduces additional bending due to the asymmetry, therefore crack grows non-uniformly through the thickness, the double-sided configuration provides much better performance than single-sided configuration. Wang et al. [43] tested CCT and SENT specimens using different in-plane CFRP configurations but with an equal stiffness ratio. The results showed that the width of the CFRP has a positive effect on fatigue life. Besides, the fatigue life extension was higher in specimens that had the composite patch initially overlapping the crack, thus allowing the bridging effect of the CFRP patch near the crack tip to be effective from the initial stages of the loading history. Similar observations were reported in ref. [35]. Commercially available CFRP laminates provided higher performance than CFRP sheets. The process of elaborating the CFRP sheets in the laboratory may introduce some additional imperfections which resulted in lower performance [43]. Liu et al. [26] investigated the effects of the patch system, thickness, length, width, and patch configurations on the fatigue behavior of single- and double-sided repaired steel plates. The fatigue results showed that CFRP with higher young modulus achieved better performance. The thickness, length, width, and patch configurations were found to have an influence on the fatigue life. The strain measurements on CFRP sheet layers demonstrated that not the full stiffness of the sheets layers is active when the specimens are loaded. Therefore, the authors adopted a method to calculate the reduction of stiffness in each sheet layer in [38]. Wu et al. [35] investigated the effect of patch configuration on the fatigue life of CCT specimens repaired with HM CFRP. It was shown that the length of the CFRP plate can influence the failure mode of the composite patch, if the length of the CFRP is smaller than the theoretical effective length proposed in [55]. Debonding of the CFRP failure mode occurred for shorter CFRP plates and fiber breakage was observed in the elliptical debonded area near the crack tip for longer CFRP plates. The length lower than the effective bond length resulted in lower performance than longer CFRP patch. It is interesting to note that HM CFRP was very efficient in extending the fatigue life where the minimum ratio was at least 3.26 times and the crack was arrested for the fully covered specimens which did not fail even after 10^8 cycles. The authors recommended to bond the CFRP patch as close as possible to the crack. The crack length of the specimen significantly influences the stress concentration thus the residual fatigue life. The effectiveness of the composite patch depends on the damage state

of the original metallic element. Some studies [24], [56]–[58] investigated the effect of the initial damage level (or initial crack length) of the metallic specimen on the performance of the CFRP repair. The results showed that regardless the crack length, the application of CFRP could extend the fatigue life. The specimens with higher initial damage had a higher ratio of increase in fatigue life. Since, initially these specimens had very short fatigue life. The number of cycles increased remains very low compared to specimens with low initial damage level. Longer cracks introduce higher stress concentration which causes debonding of the CFRP. A better crack growth retardation can be achieved when the CFRP is bonded at lower levels of damage. Earlier repair intervention was always recommended.

The review of the literature on experimental testing on CFRP crack repairs drew the following conclusions:

- The CFRP stiffness significantly influences the performance of the crack repair. It contributes to shearing the average stress, thus reducing the stress concentration. HM CFRP provides a significant stress reduction.
- The CFRP patch is more effective in reducing the stress concentration when it is overlapping the crack through the bridging effect. The distance between the location of the CFRP patch and the crack tip is an important parameter. It is recommended to cover the crack with the composite patch.
- The initial damage influences the fatigue performance of a CFRP crack repair. Due to the very short life of specimens with a high level of initial damage, the repair with CFRP patch results in a high ratio of increase in fatigue life. However, the number of cycles increased remain low compared to specimens with low initial damage. Therefore, the ratio of the increase in fatigue life may mislead the interpretation of the results. A lower crack growth retardation can be achieved when the CFRP is bonded at shorter cracks. Therefore, earlier repair intervention is recommended.
- When the length of the CFRP is lower than the effective length, it influences the performance of the crack repair. Otherwise, it has a negligible effect.
- CFRP sheets manually elaborated in the laboratory results with mechanical properties not consistent with the commercially available pultruded plates.

8.2.2 Experimental data collection

This section presents the database of 69 experimental and 7 numerical tests collected from the literature, which has been used for the development and validation of the analytical model that estimates the SIF reduction factor β . Where, 53 experimental specimens were used for the calibration of the model, and the remaining 16 experimental and 7 numerical results were used for validation. A typical bare steel CCT specimen is shown in Figure 8-2 (a). Figure 8-2 (b, c) present the different CFRP geometrical configurations that are considered in the database. Only the CFRP reinforcements that were applied in the two sides of the plates, as shown in Figure 1 (d), are considered in this study. The distance between the CFRP and notch in the steel plate was considered with (+ sign) when the CFRP was in the vicinity of the notch and with a (- sign) when the CFRP covered the notch, as shown in Figure 8-1. The mechanical and geometrical properties of CCT specimens repaired with bonded non-prestressed CFRP subjected to fatigue loading are presented in Table 8-1. The parameters E , L , W , and t represent respectively the elastic modulus, length, width, and thickness. The subscripts s denotes steel and c denotes CFRP. The full length of the crack including the diameter of the hole is denoted by $2a$. The parameter N_{cp} represents the number of CFRP patch in the face of the specimen. The number of layers of CFRP is represented by the parameter N_{cl} . The distance between the CFRP patch and the notch tip is represented by the parameter D_{nd} .

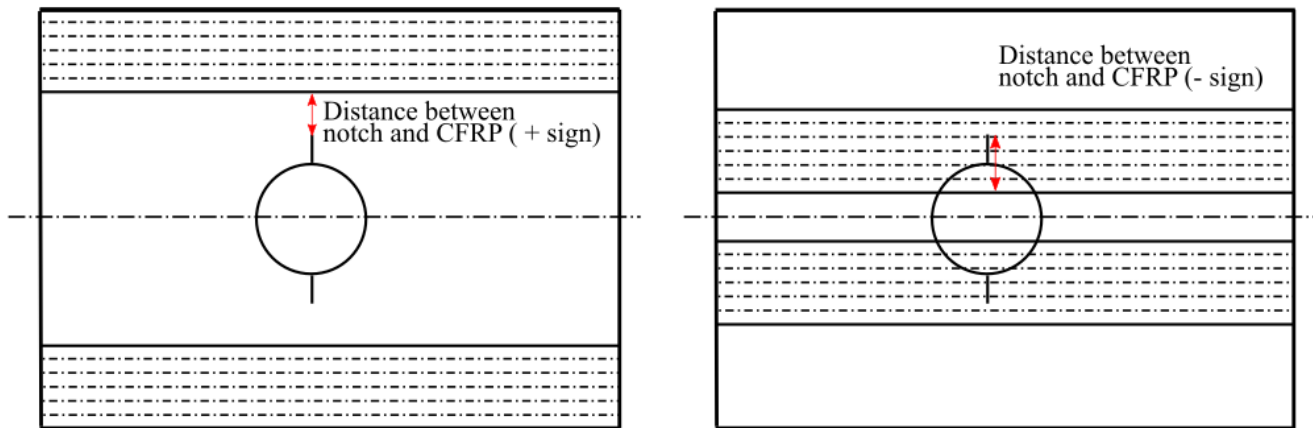
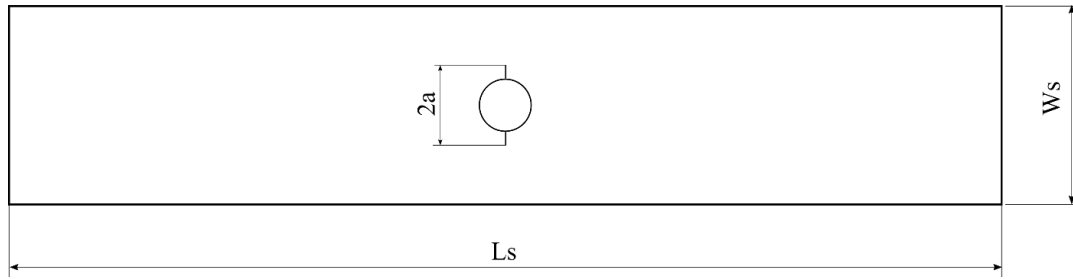
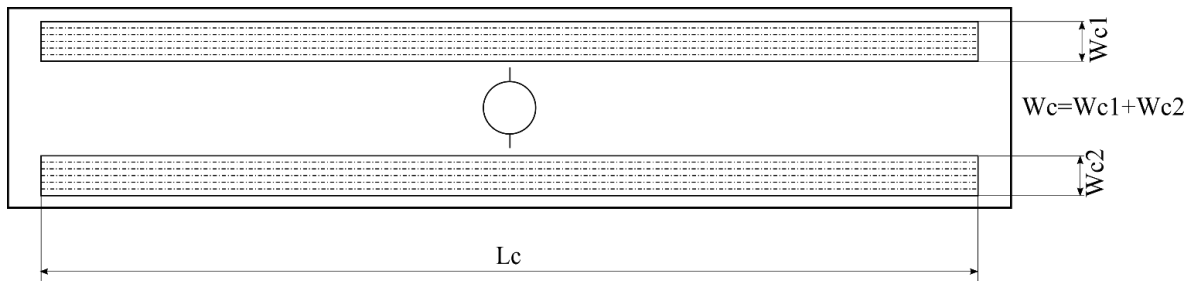


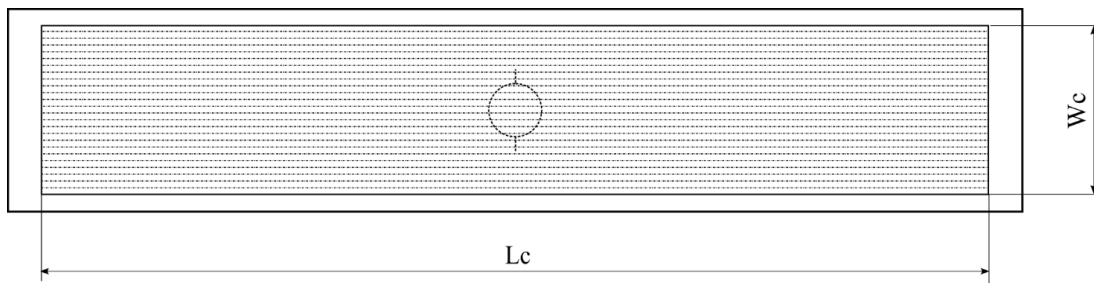
Figure 8-1: Distance between CFRP and notch.



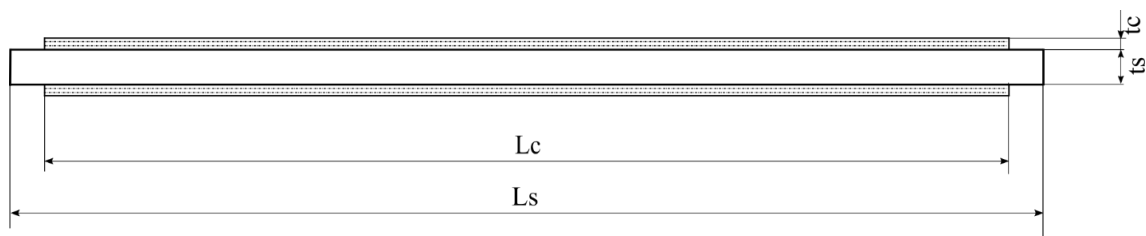
(a) Bare CCT specimen without CFRP.



(b) CCT specimen repaired with partially covered CFRP.



(c) CCT specimen repaired with fully covered CFRP.



(d) Double sided configuration (transversal view).

Figure 8-2: Geometrical characteristics of CCT specimens with and without CFRP.

Table 8-1: Geometrical and mechanical properties of the CCT specimens repaired with CFRP collected from the literature.

Use	Reference	Specimen	Steel					CFRP						
			E_s (GPa)	L_s (mm)	t_s (mm)	W_s (mm)	$2a$ (mm)	E_c (GPa)	N_{cl}	N_{cp}	t_c (mm)	W_c (mm)	L_c (mm)	D_{nd} (mm)
Calibration	[18]	Non-prestressed	200	1000	10	300	30	174	1	2	1.2	50	500	10
		Non-prestressed	200	1000	10	300	30	174	1	2	1.2	50	500	10
	[26]	DN3B250	200	500	10	90	12	240	3	1	0.176	90	500	-6
		DN5B250	200	500	10	90	12	240	5	1	0.176	90	500	-6
		DH5D250	200	500	10	90	12	640	5	1	0.19	60	500	-6
		DH5C250	200	500	10	90	12	640	5	2	0.19	30	500	0
		DH3B250	200	500	10	90	12	640	3	1	0.19	90	500	-6
		DH5B100	200	500	10	90	12	640	5	1	0.19	90	200	-6
		DH5B250	200	500	10	90	12	640	5	1	0.19	90	500	-6
	[43]	CS-1	200	700	10	120	20	165	1	1	1.4	100	500	-10
		CS-2	200	700	10	120	20	165	2	1	1.4	50	500	-10
		CS-3	200	700	10	120	20	165	1	2	1.4	50	500	0
		CS-4	200	700	10	120	20	240	6	1	0.167	100	500	-10
	[24]	N-A-2	182	500	10	90	6.8	177	1	1	1.57	90	250	-3.4
		N-A-10	182	500	10	90	14	177	1	1	1.57	90	250	-7
		N-A-20	182	500	10	90	23	177	1	1	1.57	90	250	-11.5
		N-A-30	182	500	10	90	32	177	1	1	1.57	90	250	-16
		N-A-40	182	500	10	90	41	177	1	1	1.57	90	250	-20.5
		ND-2	182	500	10	90	6.8	177	1	2	1.57	25	250	16.6
		ND-10	182	500	10	90	14	177	1	2	1.57	25	250	13
		ND-20	182	500	10	90	23	177	1	2	1.57	25	250	8.5
		ND-30	182	500	10	90	32	177	1	2	1.57	25	250	4
		ND-40	182	500	10	90	41	177	1	2	1.57	25	250	0
		HD-2	182	500	10	90	6.8	479	1	2	1.46	25	250	16.6
		HD-10	182	500	10	90	14	479	1	2	1.46	25	250	13
		HD-20	182	500	10	90	23	479	1	2	1.46	25	250	8.5
		HD-30	182	500	10	90	32	479	1	2	1.46	25	250	4
		HD-40	182	500	10	90	41	479	1	2	1.46	25	250	0
	[27]	2%	182	500	8	100	12	191	1	2	1.4	40	200	4
		10%	182	500	8	100	20	191	1	2	1.4	40	200	0
		20%	182	500	8	100	30	191	1	2	1.4	40	200	-5
	[48]	R45-0	200	500	8	90	45	681	2	1	0.143	45	250	-22.5
		R45-0-2	200	500	8	90	45	681	2	1	0.143	45	250	-22.5
	[21]	PD120a	200	700	10	100	6	165	1	1	1	100	300	-3
		PD120	200	700	10	100	6	320	1	1	1.4	100	300	-3
		PD90	200	700	10	100	6	320	1	1	1.4	100	300	-3
	[22]	D1L135	200	500	6	90	7	230	1	1	0.17	50	250	-3.5
		D2L 135	212	500	6	90	7	230	2	1	0.17	50	250	-3.5
		D3L135	212	500	6	90	7	230	3	1	0.17	50	250	-3.5
		D1L150	212	500	6	90	7	230	1	1	0.17	50	250	-3.5
		D2L150	212	500	6	90	7	230	2	1	0.17	50	250	-3.5

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Verification		D3L150	212	500	6	90	7	230	3	1	0.17	50	250	-3.5
	[35]	a-1	200	340	10	90	7	475	1	1	1.4	90	250	-3.5
		a-2	200	340	10	90	7	475	1	1	1.4	90	250	-3.5
		b-1	193	340	10	90	7	475	1	1	1.4	50	250	-3.5
		b-2	193	340	10	90	7	475	1	1	1.4	50	250	-3.5
		c-1	193	340	10	90	7	475	1	1	1.4	50	100	-3.5
		c-2	193	340	10	90	7	475	1	1	1.4	50	100	-3.5
		d	193	340	10	90	7	475	1	2	1.4	25	100	29
		e	193	340	10	90	7	475	1	2	1.4	25	100	0
	[59]	S20	227	570	10	90	7	435	2	1	0.163	90	250	-3.5
	[34]	AR-420	200	360	5	80	6.6	552	3	1	0.19	80	200	-3.3
		AR-2014	200	360	5	80	6.6	552	3	1	0.19	80	200	-3.3
	[30]	NC	205	700	8	90	9	230	2	1	0.167	45	300	-4.5
		HC-a	205	700	8	90	9	640	2	1	0.143	45	300	-4.5
		HC-b	205	700	8	90	9	640	2	1	0.143	45	300	-4.5
		HC-c	205	700	8	90	9	640	2	1	0.143	45	300	-4.5
		NC (Num)	205	700	8	90	9	230	2	1	0.167	45	300	-4.5
		HC (Num)	205	700	8	90	9	640	2	1	0.143	45	300	-4.5
	[31]	NPBR-UHM	205	900	10	150	17	435	1	2	1.5	25	400	5
	[60]	S345-195-1	201	400	16	70	7	435	3	1	0.163	70	200	-3.5
		S345-195-2	201	400	16	70	7	435	3	1	0.163	70	200	-3.5
		S460-195	201	400	12	70	7	435	3	1	0.163	70	200	-3.5
		S460-24.5-1	201	400	12	70	7	435	3	1	0.163	70	200	-3.5
		S460-24.5-2	201	400	12	70	7	435	3	1	0.163	70	200	-3.5
	[61]	SL345-195-1	201	400	16	70	7	435	1	1	0.163	50	200	-3.5
		SL460-24.5	201	400	12	70	7	435	1	1	0.163	50	200	-3.5
	[62]	SP6(CFRP_2L2S) 1	212	500	6	90	7	230	2	1	0.17	50	250	-3.5
		SP6(CFRP_2L2S) 2	212	500	6	90	7	230	2	1	0.17	50	250	-3.5
		SP5(CFRP_2L2S) 1	212	500	5	50	7	230	2	1	0.17	50	250	-3.5
		SP5(CFRP_2L2S) 2	212	500	5	50	7	230	2	1	0.17	50	250	-3.5
	[29]	ST1 (Num)	205	900	10	150	17	435	1	2	1.5	25	400	5
		ST2 (Num)	205	900	10	150	17	435	1	2	0.75	25	400	5
		ST3 (Num)	205	900	10	150	17	205	1	2	1.5	25	400	5
		ST5 (Num)	205	900	10	150	17	435	1	2	0.75	25	400	5
		ST6 (Num)	205	900	10	150	17	435	1	2	2.25	25	400	5

Note: The specimens with (Num) are numerical results.

8.3 Modelling fatigue crack growth of a repaired cracked metallic members

8.3.1 Fatigue crack growth of bare steel and CFRP repaired elements

The fatigue analysis of cracked structural details requires the knowledge of fracture mechanics and fatigue crack propagation behavior of the material. The first fatigue crack propagation law universally accepted was proposed by Paris and Erdogan [63], which correlates the fatigue crack propagation rate da/dN and the SIF range ΔK using a power relation. Then, several modified expressions have been proposed to account for the stress ratio, crack closure, opening effect, and different stages of the crack evolution, among others [64]–[67]. The crack closure, firstly observed by Elber [44], can give a pronounced effect and is one of the mechanisms behind crack arrestment. Assessment of metallic structures standards, such as BS7910 [68], allow considering the crack closure effect by means of the effective SIF range ΔK_{eff} . In this study, the model proposed by Elber [44], that accounts for crack closure, is considered (see Eq. 41). The parameters C and m are material constants, and U is the parameter that takes into account the effect of crack closure. In this study, the parameter U is estimated using the equation adopted by ASTM E 647 that assumes there is no influence of the compression part of the cyclic load on the FCG, therefore the equation comes with a set of conditions to estimate the SIF range (ΔK), as presented in Eq. 46. Further information can be found in [65], [69], [70].

$$\frac{da}{dN} = C(U\Delta K)^m \quad \text{Eq. 41}$$

The average crack growth rate is calculated using Eq. 42.

$$\left(\frac{da}{dN}\right) = \frac{a_{i+1} - a_i}{N_{i+1} - N_i} \quad \text{Eq. 42}$$

Where a is the crack length, N is the number of cycles and ΔK is the SIF range which is a function of the structural geometry, the nominal stress range, and the instantaneous crack size. The SIF is estimated using Eq. 3.

$$K = F\sigma_s\sqrt{\pi a} \quad \text{Eq. 43}$$

in which F is crack size-dependent and is given by:

$$F = F_b F_w F_g F_e \quad \text{Eq. 44}$$

where, F_b , F_w , F_g , and F_e account for the effects of the free surface, finite thickness or width, non-uniform opening stress, and elliptical crack fronts, respectively. In this study, the values of F_b , F_g , and F_e , are taken as 1.0, and F_w is estimated using Eq. 5. For more details see [38].

$$F_w = \sqrt{\sec(\pi a/b)} \quad \text{Eq. 45}$$

$$U = 0.576 + 0.015R + 0.409R^2 \quad \text{Eq. 46}$$

Where $\Delta K = K_{max}$ for $R < 0$, $\Delta K = K_{max} - K_{min}$ for $R > 0$

The composite patch will share the stress with the steel element, thus reducing the average stress in the specimen and the stress fields in the vicinity of the crack tip. Besides, when the patch overlaps the crack, a bridging effect appears between both sides of the crack, which will limit the crack opening [36]. The relationship between the fatigue crack growth rate and the stress intensity factor in the repaired specimen obeys the same FCG relationship as that of the unrepaired specimen [71]. The effect of the composite patch in the fatigue crack growth model is commonly taken into account using some parameters that will reduce the SIF range ΔK [36], [42]. In this study, a SIF reduction factor β is evaluated based on experimental data that are shown in the previous section. An analytical equation to estimate β is derived using nonlinear regression analysis by considering the most influencing parameters as it will be shown in the subsequent sections. The modified fatigue propagation model that includes the SIF reduction factor β is presented in Eq. 7.

$$\frac{da}{dN} = C (\beta U \Delta K)^m \quad \text{Eq. 47}$$

The number of cycles to failure of a cracked member can be estimated using the following expression:

$$N_f = \int_{a_i}^{a_f} 1/(C (\beta U \Delta K)^m) da \quad \text{Eq. 48}$$

8.3.2 Procedure for the evaluation of the SIF reduction factor β

The procedure used in this study to calibrate the model is the following:

- (1) Calibrate the fatigue crack growth model for unrepaired steel specimens using experimental data. Thus, the material constants C is obtained for each test. Note that the exponential component m was assumed

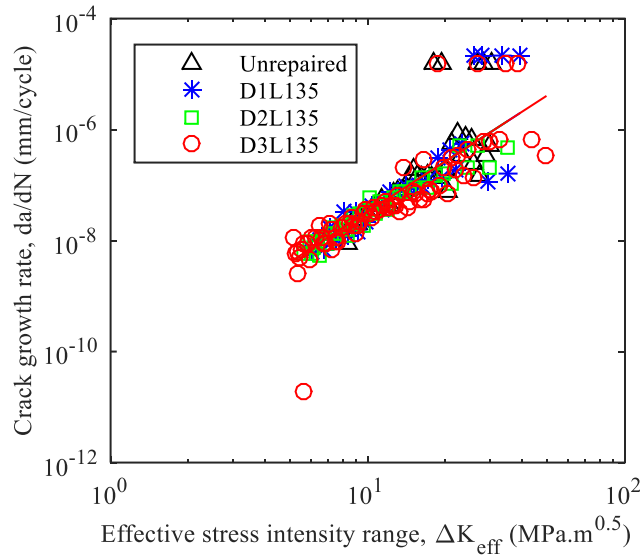
constant for all tests and equals to $m = 3$, as in BS7910 [68]. This is an important hypothesis since β may be influenced by the value of m .

- (2) Then, the effect of the CFRP patch could be evaluated by fitting the fatigue crack growth curves for repaired specimens, using Eq. 47, where the reduction factor β is the variable.
- (3) Once β is obtained for all experimental results, an analysis of the trends of the data was conducted to identify the most influencing parameters, as shown in the following section.
- (4) Finally, multiple nonlinear regression analysis is performed to derive an analytical law that estimates β . The regression variables included in the equation are those that influenced the experimental results. This includes parameters related to the CFRP stiffness, initial damage of the specimen, and the location of the CFRP with respect to the crack tip.

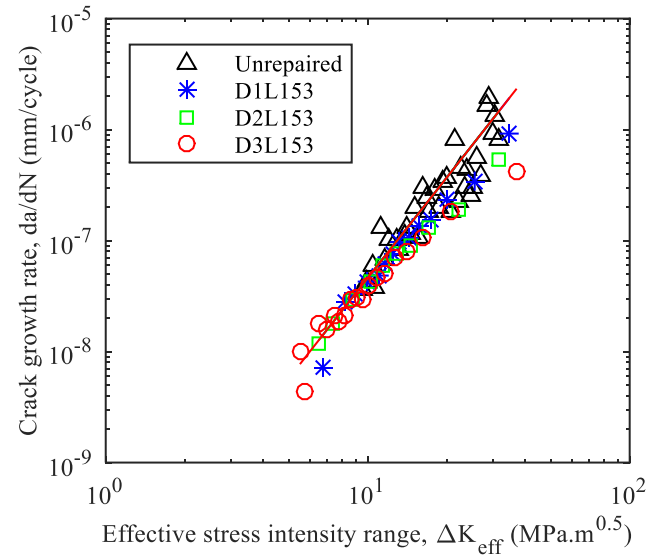
8.3.3 Calibration results of the fatigue crack growth model

This section presents the calibration results of the fatigue crack growth model for unrepaired and repaired specimens. This consists of steps (1) and (2) of the procedure presented in the previous section. The parameters C and m are determined by fitting Eq.1 to crack growth rate data of unrepaired specimens. The relationship between the fatigue crack growth rate and the SIF in the repaired and unrepaired specimens is assumed to obey the same fatigue crack growth relationship [71]. Therefore, C and m are the same for unrepaired and repaired specimens. The effect of the CFRP reduces the SIF through β which is obtained by fitting Eq.7 to crack growth rate data of repaired specimens. The results of the calibration for all specimens are presented in Table 8-2. The stress range and stress ratio of the fatigue loading are denoted by $\Delta\sigma$ and R .

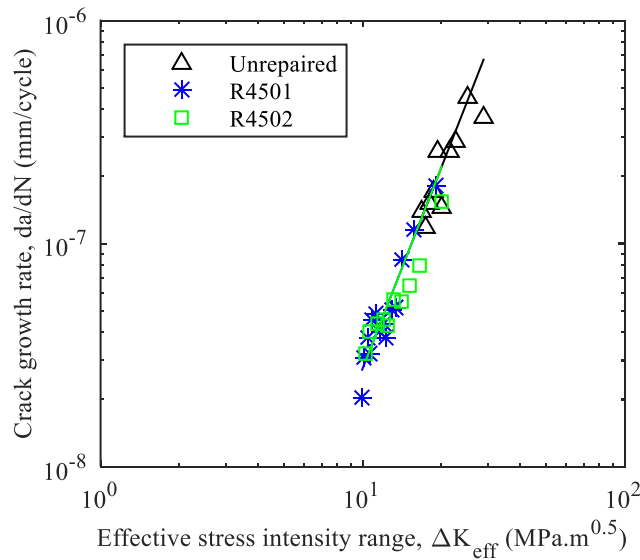
Figure 8-3 illustrates the fatigue crack growth rate versus the effective SIF range of unrepaired and repaired specimens. Both repaired and unrepaired specimens follow the same crack growth trend, which confirms the assumption adopted in this study. Figure 8-4 illustrates crack length evolution with respect to the number of cycles for unrepaired and repaired data. The model proposed in Eq.7 that includes the SIF reduction factor β predict fairly well the crack evolution.



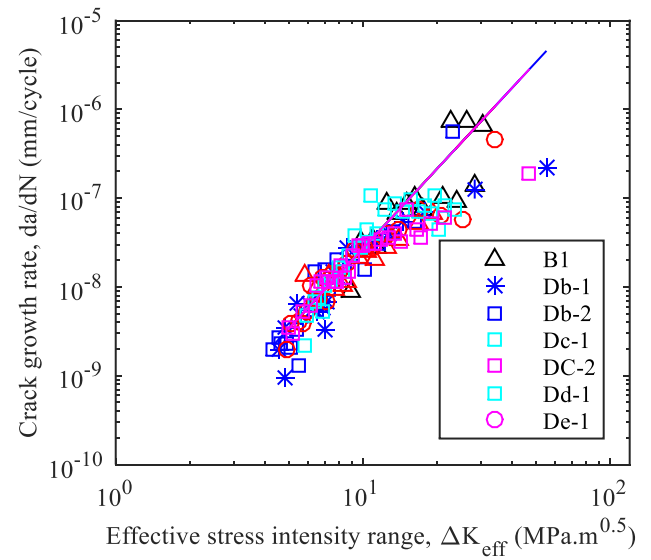
(a)



(b)



(c)



(d)

Figure 8-3: Fatigue crack growth rate versus the effective SIF range for unrepaired and repaired specimens : (a) Emdad et al. [22] ($\Delta\sigma = 135\text{MPa}$); (b) Emdad et al. [22] ($\Delta\sigma = 153\text{MPa}$); (c) Chen et al. [48]; (d) Wu et al. [35].

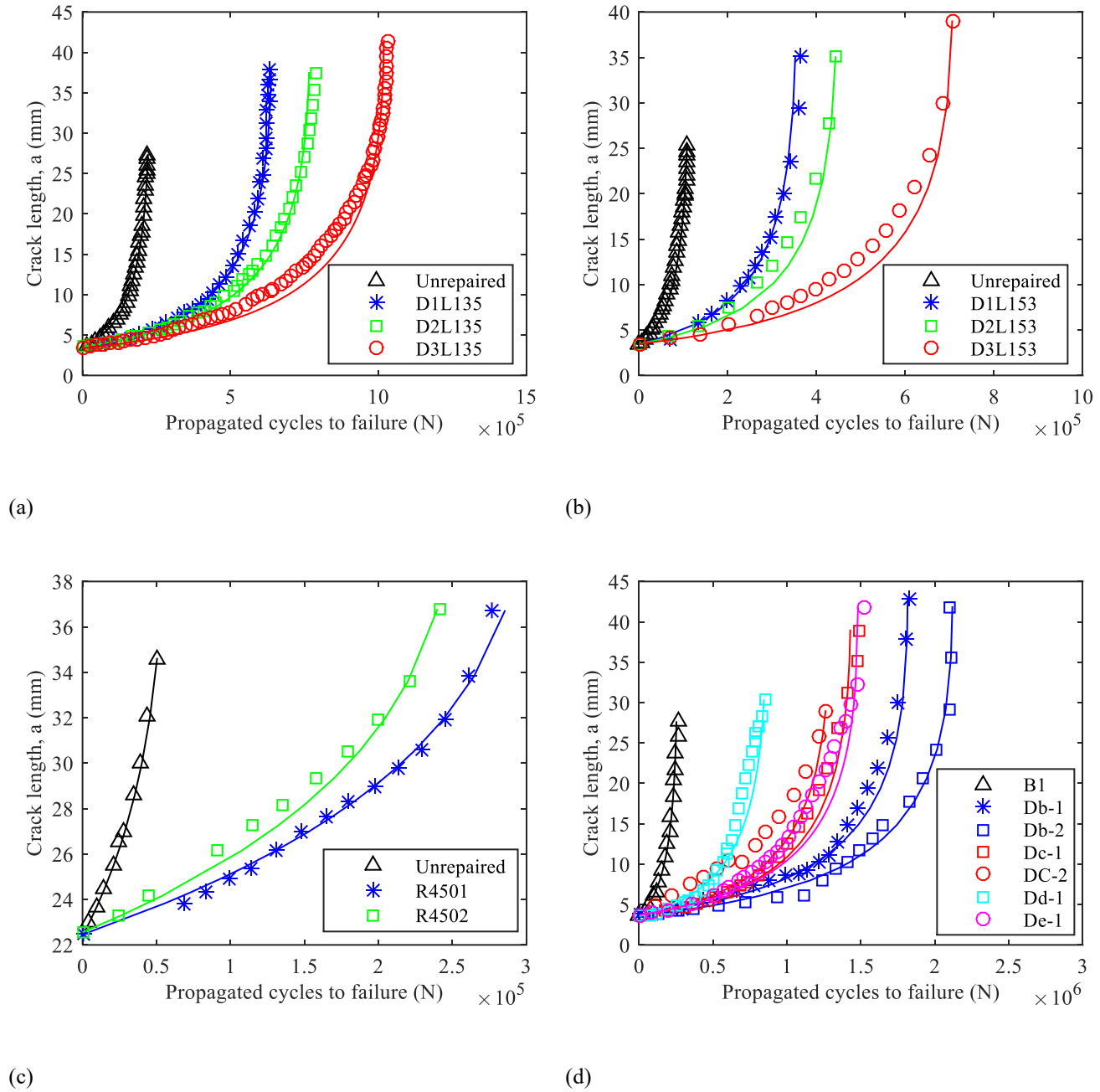


Figure 8-4: Experimental and predicted Crack length versus cycles to failure: (a) Emdad et al. [22] ($\Delta\sigma = 135\text{MPa}$); (b) Emdad et al. [22] ($\Delta\sigma = 153\text{MPa}$); (c) Chen et al. [48]; (d) Wu et al. [35].

Table 8-2: Fatigue life results and crack growth rate parameters of the experimental data used for the calibration of the model.

References		Fatigue loading		Cycles to failure		Steel		
		$\Delta\sigma$ (MPa)	R	unrepaired	repaired	C	m	β
[18]	non-prestressed	80	0.4	3.00E+05	9.00E+05	3.40E-11	3	0.63
[26]	DN3B250	150	0.1	2.42E+05	5.32E+05	2.10E-11	3	0.78
	DN5B250	150	0.1	2.42E+05	6.52E+05	2.10E-11	3	0.73
	DH5D250	150	0.1	2.42E+05	1.14E+06	2.10E-11	3	0.61
	DH5C250	150	0.1	2.42E+05	1.21E+06	2.10E-11	3	0.6
	DH3B250	150	0.1	2.42E+05	1.59E+06	2.10E-11	3	0.54
	DH5B100	150	0.1	2.42E+05	1.88E+06	2.10E-11	3	0.52
	DH5B250	150	0.1	2.42E+05	1.91E+06	2.10E-11	3	0.51
[43]	CS-1	120	0.2	1.37E+05	6.73E+05	2.85E-11	3	0.601
	CS-2	120	0.2	1.37E+05	5.49E+05	2.85E-11	3	0.64
	CS-3	120	0.2	1.37E+05	4.81E+05	2.85E-11	3	0.67
	CS-4	120	0.2	1.37E+05	5.91E+05	2.85E-11	3	0.62
[24]	N-A-2	135	0.1	2.04E+05	1.41E+06	3.80E-11	3	0.53
	N-A-10	135	0.1	9.24E+04	8.08E+05	3.80E-11	3	0.48
	N-A-20	135	0.1	4.80E+04	4.22E+05	3.80E-11	3	0.45
	N-A-30	135	0.1	2.59E+04	4.47E+05	3.80E-11	3	0.36
	N-A-40	135	0.1	2.59E+04	7.60E+05	3.80E-11	3	0.31
	ND-2	135	0.1	2.04E+05	3.67E+05	3.80E-11	3	0.80
	ND-10	135	0.1	9.24E+04	2.13E+05	3.80E-11	3	0.78
	ND-20	135	0.1	4.80E+04	1.20E+05	3.80E-11	3	0.74
	ND-30	135	0.1	2.59E+04	8.28E+04	3.80E-11	3	0.70
	ND-40	135	0.1	2.59E+04	1.37E+05	3.80E-11	3	0.63
	HD-2	135	0.1	2.04E+05	6.32E+05	3.80E-11	3	0.68
	HD-10	135	0.1	9.24E+04	3.60E+05	3.80E-11	3	0.66
	HD-20	135	0.1	4.80E+04	2.21E+05	3.80E-11	3	0.62
	HD-30	135	0.1	2.59E+04	1.60E+05	3.80E-11	3	0.56
	HD-40	135	0.1	2.59E+04	2.61E+05	3.80E-11	3	0.50
[27]	2%	110	0.1	2.35E+05	4.63E+05	3.80E-11	3	0.78
	10%	110	0.1	1.24E+05	2.35E+05	3.80E-11	3	0.81
	20%	110	0.1	6.56E+04	1.88E+05	3.80E-11	3	0.71
[48]	R45-0	95	0.1	5.35E+04	2.92E+05	2.75E-11	3	0.51
	R45-0-2	95	0.1	5.35E+04	2.62E+05	2.75E-11	3	0.54

[21]	PD120a	120	0.4	1.82E+05	1.00E+06	3.15E-11	3	0.66
	PD120	120	0.4	1.82E+05	6.50E+05	3.15E-11	3	0.57
	PD90	90	0.4	4.84E+05	3.31E+06	2.80E-11	3	0.53
[22]	D1L135	135	0.1	2.19E+05	6.33E+05	3.30E-11	3	0.71
	D2L 135	135	0.1	2.19E+05	7.87E+05	3.30E-11	3	0.66
	D3L135	135	0.1	2.19E+05	1.03E+06	3.30E-11	3	0.60
	D1L150	153	0.1	1.33E+05	3.63E+05	4.60E-11	3	0.68
	D2L150	153	0.1	1.33E+05	4.41E+05	4.60E-11	3	0.63
	D3L150	153	0.1	1.33E+05	7.03E+05	4.60E-11	3	0.54
[35]	a-1**	135	0.1	2.63E+05	1.00E+08	2.70E-11	3	0.12
	a-2**	135	0.1	2.63E+05	1.00E+08	2.70E-11	3	0.12
	b-1	135	0.1	2.63E+05	1.83E+06	2.70E-11	3	0.52
	b-2	135	0.1	2.63E+05	2.11E+06	2.70E-11	3	0.50
	c-1	135	0.1	2.63E+05	1.49E+06	2.70E-11	3	0.57
	c-2	135	0.1	2.63E+05	1.25E+06	2.70E-11	3	0.59
	d	135	0.1	2.63E+05	8.58E+05	2.70E-11	3	0.68
	e	135	0.1	2.63E+05	1.53E+06	2.70E-11	3	0.56
[59]	S20	148.5	0.1	3.52E+05	1.24E+06	4.80E-11	3	0.72
[34]	A-420**	135	0.1	2.49E+05	5.98E+06	2.75E-11	3	0.34
	A-2014**	135	0.1	2.49E+05	5.98E+06	2.75E-11	3	0.34

Note: The specimens with superscript (**) are run-outs.

8.3.4 Trends of The SIF reduction factor β

This section presents the trends of the SIF reduction factor β versus the different parameters that may influence the fatigue performance of cracked elements repaired with CFRP, as discussed before in the review of past experimental observations. The parameters considered are as following:

- The stiffness ratio: which is the ratio between the CFRP stiffness and the net cross-section of the steel plate. The diameter of the hole is deduced to calculate the section but the initial crack length is not included. This will determine the contribution of the CFRP in sharing the average stress in the net cross-section. The effective stiffness of manually fabricated CFRP sheets was considered based on the work of Liu et al. [38].

$$R_s = (n_c \times w_c \times t_c \times E_c) / (w_s \times t_s \times E_s) \quad \text{Eq. 49}$$

- The initial damage ratio: which will include the effect of the initial damage level on the crack growth behavior.

$$R_{id} = 2a/w_s \quad \text{Eq. 50}$$

- The distance CFRP-notch ratio: which indicates the distances between the crack tip and the CFRP plate in the transversal direction of the specimen, as shown in Figure 8-1. Note, the origin of the reference axis is taken at the crack tip, therefore, if the CFRP covers the crack, the distance is taken with a minus sign.

$$R_{nd} = \pm D_{nd} / (0.5 \times (W_s - 2a)) \quad \text{Eq. 51}$$

- Length ratio: which represents the proportion of the CFRP length with respect to the length of the specimen.

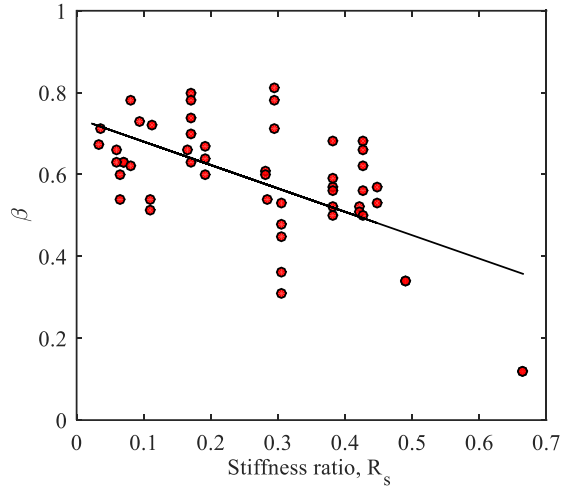
$$R_l = L_c / L_s \quad \text{Eq. 52}$$

Figure 8-5 (a) illustrates the variation of β versus the stiffness ratio " R_s " that was determined according to Eq.9. Clearly, an increase in the stiffness ratio is associated with a decrease of β . Since, β is in the denominator of Eq.8, a decrease of β implies an increase in the fatigue life. Therefore, the trend is consistent with the observations of the previously discussed experimental results regarding the effect of stiffness ratio. Figure 8-5 (b) shows the trend

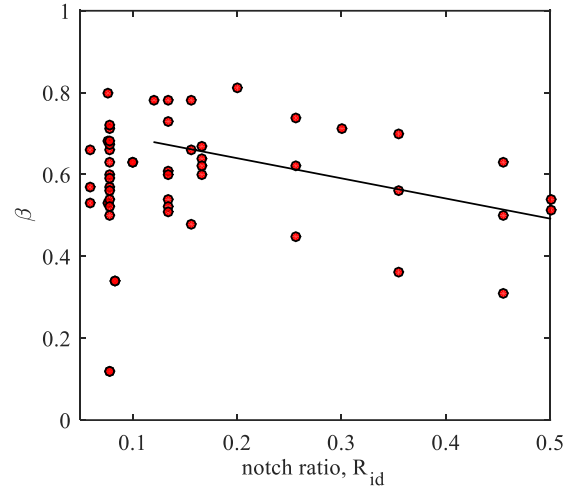
of β versus the initial damage ratio " R_{id} " determined according to Eq.10. The factor β tends to decrease with increasing the R_{id} . This is in line with the experimental observations, where the ratio of increase in fatigue life is higher for specimens with high initial damage. Figure 8-5 (c) illustrates the variation of β versus the notch distance ratio " R_{nd} " which represents the location of the CFRP patch with respect to the initial notch tip. R_{nd} was calculated using Eq. 11. The results show a clear influence of the location of the CFRP on the performance of the CFRP repair, where, a lower R_{nd} (i.e. the closer the CFRP to the crack tip) provides lower values of β . Thus, increasing the fatigue life. As mentioned previously, the CFRP patch provides a bridging effect that limits crack opening when the CFRP covers the crack. Therefore, the closer the patch to the initial crack tip, the better performance will be achieved. The CFRP-notch distance is estimated as presented in Figure 8-1. The minus sign (-) indicates that the CFRP is covering the crack. In the case of fully covered configuration, the distance is taken between the notch tip and the center of the crack, thus it becomes equal to half initial crack length " a ". Figure 8-5 (d) shows large scattering when β is plotted against the length ratio ($R_l = L_c/L_s$) without showing a clear indication of its influence. Therefore, the effect of the CFRP length will be not considered in the development of the predictive model. The scattering of the data is mainly due to the limited experimental tests that studied the effect of the CFRP length, and as discussed previously, the CFRP length has a negative influence only if it was lower than the effective theoretical length threshold. Longer CFRP was implemented in most experimental tests.

8.3.5 Regression analysis

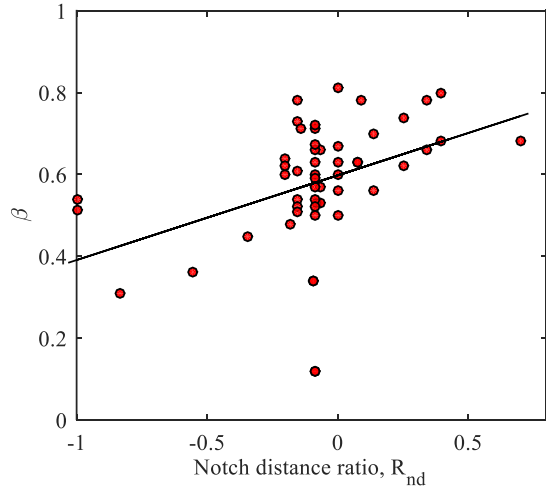
In this section, a regression equation is proposed to predict the SIF reduction factor β based on the previously discussed parameters. The fitting was conducted using 70% (53 specimens) of the database based on random selection. The remaining data were used for the validation of the model, as it is shown in section 6.1. A nonlinear regression model is used to take into account the contribution of each parameter discussed in the previous section. This includes the stiffness ratio R_s , the initial damage ratio R_{id} and the notch-CFRP distance ratio R_{nd} . A quadratic



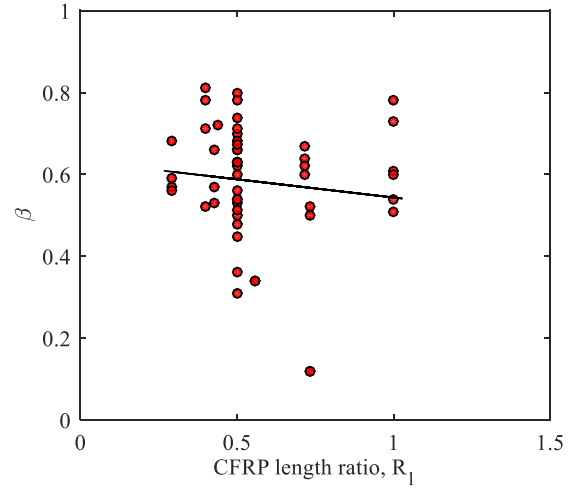
(a)



(b)



(c)



(d)

Figure 8-5: Dependence of the SIF reduction factor β on: (a) on the CFRP-steel stiffness ratio; (b) CFRP-steel width ratio; (c) the ratio that describes the location of the CFRP with respect to the notch tip; (d) CFRP-steel length ratio.

polynomial, as shown in Eq.13, was found to be the most appropriate to account for the combined contributions of each parameter. This type of mathematical model was also adopted in recent studies as in [45]. The calibrated parameters α_i and β_i of the regression model are presented in Eq.14. Note that 4 run-outs were included in the regression analysis, which will provide conservative results since the specimens did not fail. It was found important to consider these run-outs to capture the effect of stiffness for high modulus CFRP.

$$\beta = \left(\sum_i^n \alpha_i R_i + \beta_i R_i^2 \right) \quad \text{Eq. 53}$$

$$\beta = (0.3964R_s + 0.1337R_{id} + 0.2701R_{nd} - 1.7563R_s^2 - 0.2892R_{id}^2 + 0.0473R_{nd}^2 + 0.6548) \quad \text{Eq. 54}$$

$$R^2 = 0.79 \text{ and } \sigma = 0.127$$

The experimental data used to derive this equation are within the following range of parameters:

$$0.034 \leq R_s \leq 0.665$$

$$0.06 \leq R_{id} \leq 0.5$$

$$-1.0 \leq R_{nd} \leq 0.7$$

The R-squared is estimated by 0.79 and the standard deviation of the ratio between predicted and experimental β is equal to $\sigma = 0.127$. A probabilistic analysis of the model is highly recommended to provide more reliable design equations. Figure 8-6 shows the fitting graphical test of predicted and experimental β . Most of the data is bounded by two lines of $\pm 20\%$ of error. Figure 8-7 illustrates the comparison between predicted and experimental fatigue life. As it can be seen, the proposed model provides a fair estimation of the fatigue life of the experimental results.

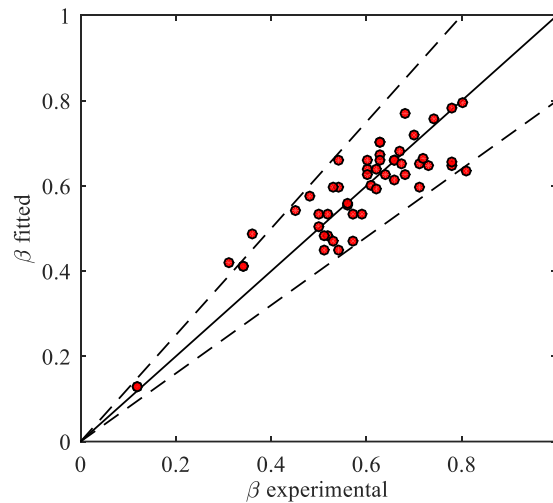


Figure 8-6: Fitting of SIF reduction factor β .

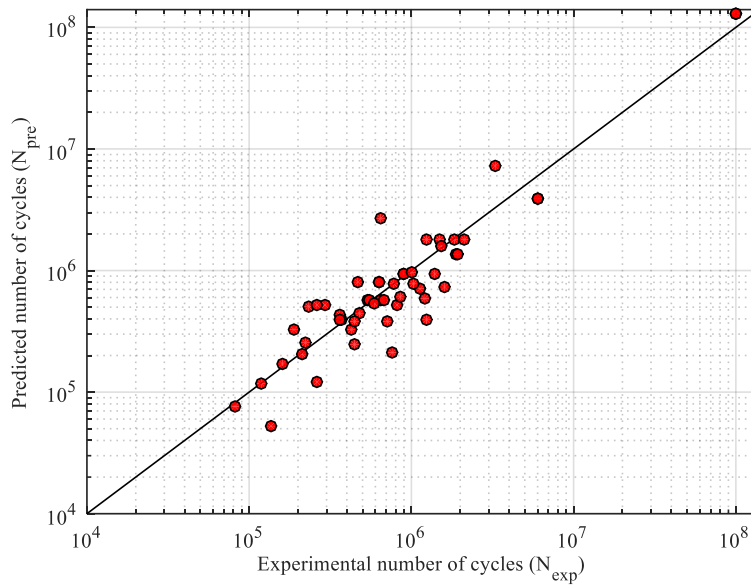
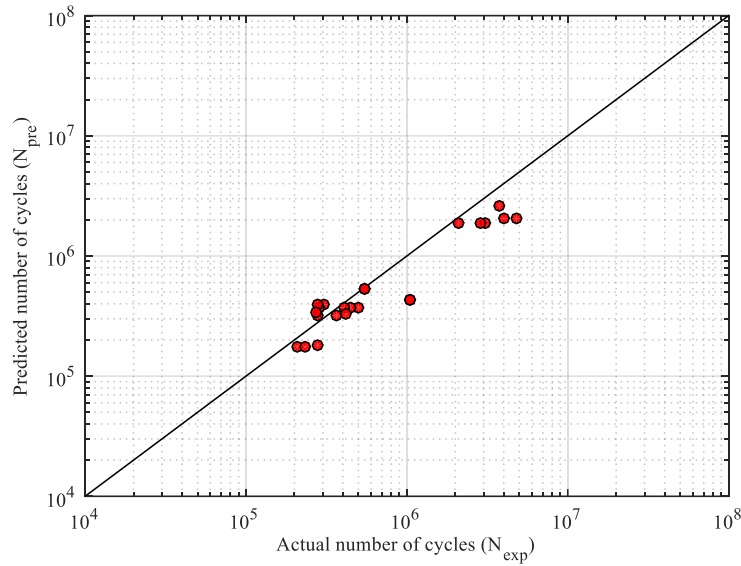


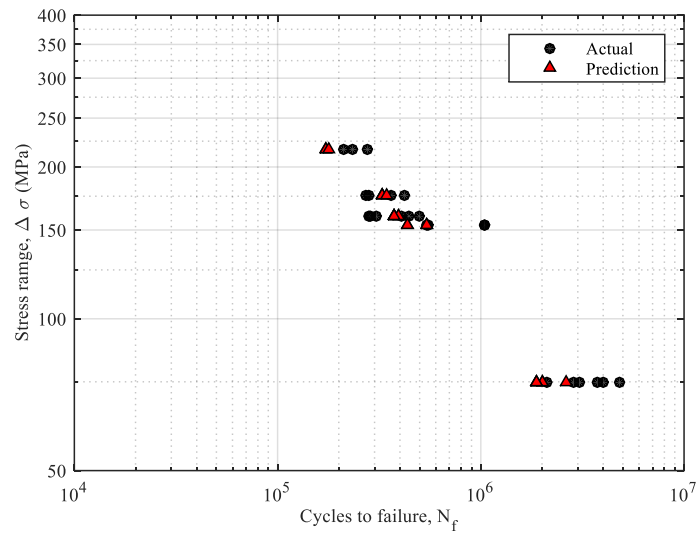
Figure 8-7: Number of cycles to failure: predicted vs. experimental.

8.3.6 Validation of the model

As stated previously, the model proposed in Eq.14 was calibrated with 70% of the data presented in Table 1. The fatigue life results of the remaining 30% of data are used for the validation only. The validation is conducted by assuming that the fatigue crack propagation parameters of the steel can be assumed equal to $C = 3.3 \times 10^{-11}$ and $m = 3$, which are the averages obtained from the experimental data used for the calibration of the model. The parameter “ β ” is calculated using the mechanical and geometrical characteristics of the specimens as presented in Table 8-1. Figure 8-8 (a) shows the comparison between the predicted fatigue life and the experimental and numerical data (Actual data) used for verification. It can be seen that the predicted fatigue life approximates fairly well the data used for verification. The standard deviation of the ratio between the predicted and actual number of cycles is equal to $\sigma = 0.29$. Figure 8-8 (b) shows the stress range versus the number of cycles for both predicted and actual data. It shows that Eq.14 approximates the fatigue life fairly well for different stress range levels.



(a) Predicted number of cycles VS Experimental and numerical number of cycles.



(b) Comparison between fatigue results predicted using Eq.14 and actual data used for verification.

Figure 8-8: Validation of the proposed model.

8.4 Parametric study

The sensitivity of the parameters on the predictive model proposed in Eq. 54 is presented in this section. First, a general behavior predicted by the model for a given specimen with different stress ranges is presented, then the influence of each parameter of the model is investigated independently.

Let's assume a steel CCT specimen (see Figure 8-2) subjected to fatigue loading with stress ratio $R = 0.1$. The specimen dimensions are: $t_s = 10\text{mm}$, $W_s = 100\text{mm}$, $L_s = 500\text{mm}$, and a crack length $2a = 20\text{mm}$. The elastic modulus of the steel is equal to $E_{steel} = 200\text{GPa}$. The fatigue crack propagation parameters of the steel can be assumed equal to $C = 3.3 \times 10^{-11}$ and $m = 3$, which are the averages of the experimental data. The specimen is repaired with CFRP on both sides and with a partially covered configuration, as in Figure 8-2 (b). The dimensions of each CFRP plate are $L_c = 400\text{mm}$, $W_c = 30\text{mm}$, and $t_c = 1.4\text{mm}$. The CFRP plates are located at 5mm distance from the crack tip. The modulus of elasticity of the CFRP equal to $E_c = 250\text{GPa}$. Figure 8-9 shows the evolution of the SIF for different stress ranges versus the crack length for the bare steel CCT and the repaired one. The stress range is varied between 80-200MPa. From Figure 8-9 (b), the stress intensity factor reduces considerably due to the repair with CFRP.

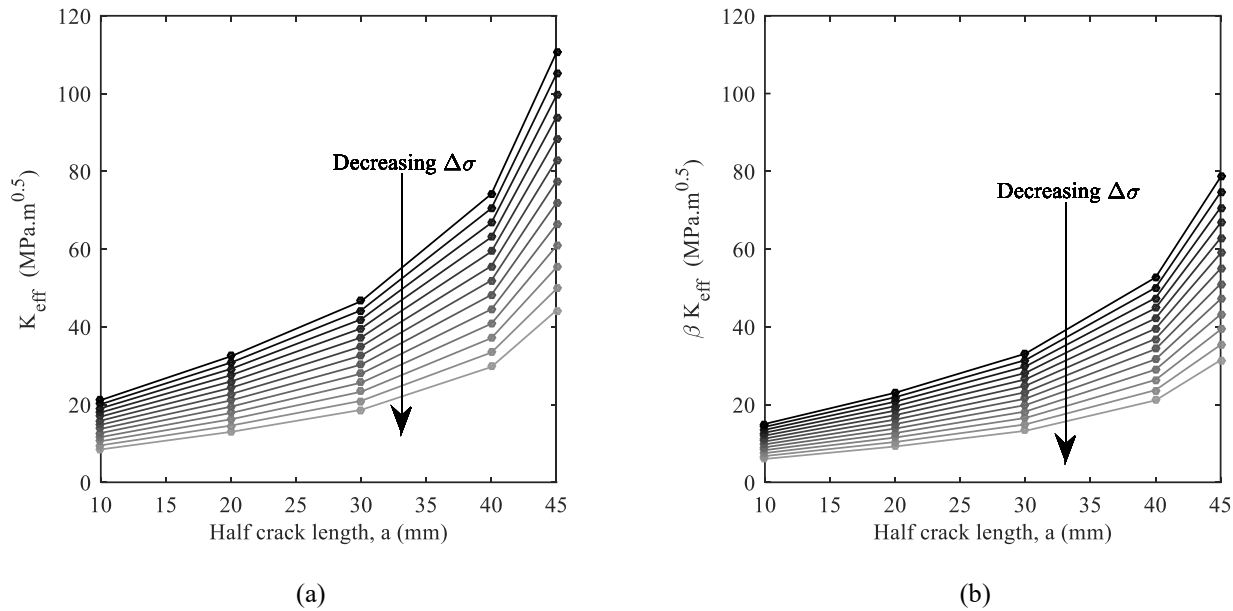


Figure 8-9: Reduced effective stress intensity factor versus half crack length: (a) bare steel CCT specimen; (b) CCT repaired with CFRP.

8.1.1.1 Effect of stiffness ratio (R_s)

In this subsection, the influence of the stiffness ratio on Eq. 54 is studied by varying it between $0.1 \leq R_s \leq 0.8$, and keeping the other parameters constant: stress range $\Delta\sigma = 150\text{MPa}$, the initial damage ratio equals to $R_{id} = 0.1$, and assuming the CFRP covers the notch with a CFRP-notch ratio equals to $R_{nd} = -0.1$. The results are

presented in Figure 8-10, where the fatigue life is considerably increased after the repair with CFRP. The fatigue life is increasing in an exponential trend while the CFRP-steel stiffness ratio increases. This is consistent with experimental observations where run-outs were achieved when high modulus CFRP was applied.

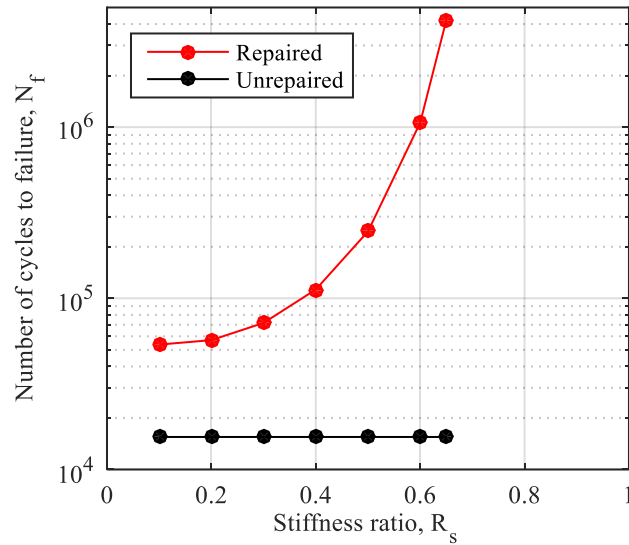


Figure 8-10: Influence of the CFRP stiffness ratio on fatigue life.

8.1.1.2 Effect of initial damage ratio (R_{id})

The influence of the initial damage ratio R_{id} on the prediction of the fatigue life of the CCT specimen using Eq. 54 is investigated in this subsection. The initial damage ratio is varied between $0.01 \leq R_{id} \leq 0.6$ and the other parameters are kept constant where $\Delta\sigma = 150$ MPa, $R_s = 0.3$, and the $R_{nd} = -0.1$. Figure 8-11 shows that the fatigue life for both repaired and unrepaired decreased with the increase of R_{id} . Regardless the initial crack length, an increase of fatigue life for repaired specimens compared to unrepaired specimens can be observed. However, for cases with higher values of R_{id} , the number of cycles increased is very low compared to the cases with lower R_{id} . Therefore, the increase of the fatigue life for specimens with a high level of initial degree of damage may become insignificant. So, as previously discussed, earlier CFRP repair is recommended.

8.1.1.3 Effect of distance CFRP-notch ratio

The influence of the location of the CFRP patch with respect to the notch tip on the fatigue life of the CCT specimen is investigated in this subsection. The CFRP-notch ratio is assumed to vary between $-0.5 \leq R_{nd} \leq 0.4$.

The other parameters are kept constant where $\Delta\sigma = 150$ MPa, $R_s = 0.3$ and $R_{id} = 0.1$. referring to Figure 8-12, the fatigue life tends to decrease with increasing the distance. This is in line with the experimental results discussed previously. The CFRP is much more efficient when it covers the initial crack (i.e. distance with (-) sign), which allows bridging the two sides near the crack tip, which will limit crack opening from the early stages of crack propagation life.

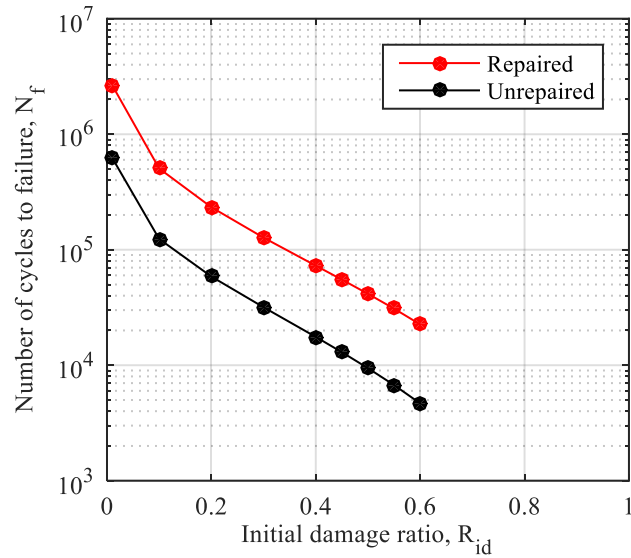


Figure 8-11: Influence of the initial damage ratio on fatigue life.

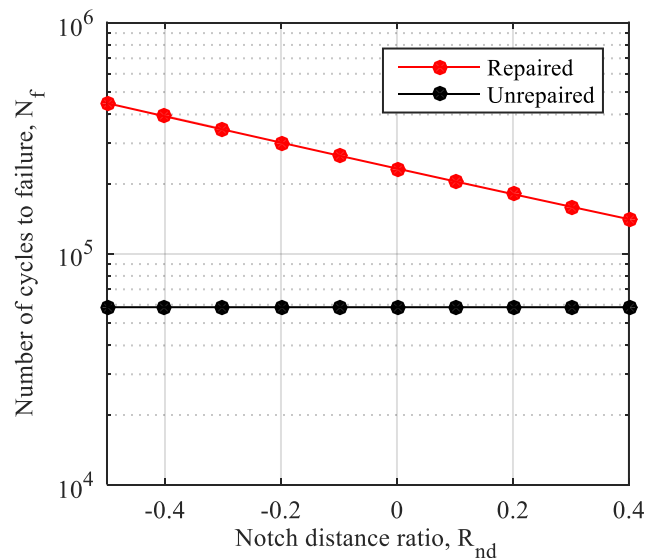


Figure 8-12: Influence of the CFRP patch location on the fatigue life.

8.5 Conclusions

This chapter presents a model to predict the fatigue crack growth of CCT specimens repaired with CFRP bonded on the two sides of the plate. The model is based on experimental data collected from the literature.

Firstly, a review of the behaviors observed in past experimental tests is presented. It was shown that the CFRP-Steel stiffness ratio significantly influences the performance of the CFRP repair. The patch location also influences the fatigue life of the repaired specimen. The length of the CFRP has a considerable influence only if it is lower than the theoretical effective length. The level of initial crack length also called damage level influences considerably the performance of the CFRP repair. The sheets manually fabricated in the laboratory achieve lower performance than pultruded ones commercially available.

A database of 69 experimental and 7 numerical CCT repaired with CFRP was developed based on the data available in the literature. The database includes the geometrical and mechanical properties of the CCT specimens and the CFRP patches, as well as the details of the fatigue loading and the crack propagation model parameters. The fatigue crack growth model that takes into account the effect of crack closure is fitted to unrepaired specimens. The effect of the CFRP repair is taken into account through a SIF reduction factor β that reduces the stress intensity factor.

The dependence of β on the parameters that influenced the fatigue life in past experimental tests was clearly shown. An analytical law was developed using nonlinear regression analysis to evaluate the β based on the CFRP stiffness ratio, the initial damage level, and the patch location. The model fits fairly well the experimental results and follow reasonably the trends for each parameter.

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**Part III: Application of CFRP to extend the deformation
capacity of steel members subjected to cyclic loading for
the seismic design of steel structures**

**CHAPTER 9: Deformation capacity of I steel beams
subjected to cyclic loading for the seismic design of
structures**

9 Deformation capacity of I-shaped steel beams subjected to cyclic loading for the seismic design of steel structures

Abstract

The provisions of the European seismic assessment code Eurocode 8 part 3 for steel members part of the seismic moment-resisting frames were the subject of a lot of discussion in the literature. Experimental evidence showed that ductile I-shaped steel beams reach very low deformation capacity when subjected to cyclic loading, which is around 30% of the deformation capacity under monotonic loading adopted by design codes. This chapter focuses on the characterization of the deformation capacity limits of steel deep and shallow beams with European profiles subjected to cyclic loading by considering several influencing parameters such as geometrical imperfections, material ductility, lateral slenderness, and member length. For this purpose, an advanced finite element model validated with past experimental data was developed and an extensive parametric study was conducted. The numerical simulations revealed that shallow and deep beams have different buckling behaviors and rotation capacity trends. Based on the numerical results and the experimental data collected from the literature, empirical equations are derived to quantify the plastic rotation limits. Compared with existing estimations, the proposed empirical equations show a more reasonable estimate. These rotation limits can be useful as modelling parameters for pushover analysis and for the identification of the damage states in the context of performance-based earthquake engineering.

9.1 Introduction

9.1.1 Context

Steel moment-resisting frames are commonly adopted in earthquake-prone regions due to their ability to dissipate energy through plastic hinges in pre-defined locations such as beam ends. High ductility class hot-rolled steel profiles (i.e. class 1) according to Eurocode 3 [1] should be able to develop large plastic deformation before significant loss of strength in an earthquake event [2]. However, recent

research studies showed that this is not necessarily the case when they are subjected to cyclic loading [2], [3]. As shown in Figure 9-1, Araujo et al. [4] showed that the Eurocode 8 part 3 rotation limits (i.e. DL, SD, and NC, see section 9.2 for definitions) are over-estimated when compared with experimental results under cyclic loading conditions. This issue has been the subject of a lot of discussion in the literature [5]–[7]. Currently, there is an urgent need to characterize the rotations capacity of steel members subjected to cyclic loading for the new generation of Eurocodes, and propose a retrofit solution for existing structures with members showing poor performance. This chapter focuses on the characterization of the rotation capacity limits of steel I-shaped members commonly used in the construction industry as beams part of seismic moment resisting frames.

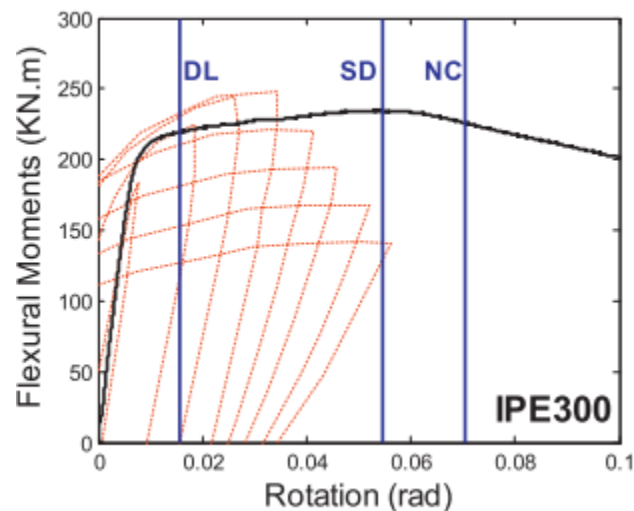


Figure 9-1: Comparison between experimental results and Eurocode 8 part rotation limits for class 1 ductile beams [4].

9.1.2 Background

The deformation capacity limits also called rotation capacity for members under flexural loading, are the key parameters to describe the nonlinear behavior of steel beams part of seismic moment resisting frames. Seismic assessment codes such as Eurocode 8 part 3 (EC8-3) [8] and ASCE41 [9] adopted the performance-based earthquake engineering (PBEE) philosophy, where plastic rotation limits corresponding to different

Chapter 9: Deformation capacity of I steel beams subjected to cyclic loading for the seismic design of structures

damage states are provided. The plastic rotation capacity limits obtained from cyclic loading can be adopted as acceptance criteria for design checks as well as modelling parameters for numerical models where the cyclic deterioration is not considered as in the case of pushover analysis. The reliability of the of the analysis will depend strongly on the accuracy of the plastic rotations. Therefore, providing more accurate estimation of the plastic rotations of steel beams is of paramount interest.

With recent advances in computing, seismic design and assessment codes are increasingly improving the provisions for nonlinear analysis methods. Therefore, design engineers are typically required to use code-based modelling and acceptance criteria guidelines. However, several researchers and engineering practitioners (Hartloper and Lignos, [10], [11], Bech *et al.* [12]., Araujo *et al.*, [4], El Jisr *et al.*, [13], Lignos and Hartloper [14]) have identified some inconsistencies between state of the art experimental results and code based rotation limits in ASCE41-13 and EC8-3. In European context, the EC8-3 provision for steel beams and beam-columns are very limited compared to ASCE41 and has been the subject of lot of discussion in the literature. Araujo *et al.*, [4] highlighted the limitations of the EC8-3 provisions and showed that the EC8-3 rotations are much larger than the experimental results for IPE beam tested by D'Aniello *et al.*, [2]. Similarly, El Jisr *et al.*, [13] found that the EC8-3 over-estimates the rotation capacity of composite steel beams by 50% regardless the web slenderness ratio. Therefore, EC8-3 provisions for steel moment frames requires some important revisions in the upcoming version.

Araujo *et al.*, [4] conducted an extensive finite element study to estimate the rotations of steel members with wide range of I and H shaped European profiles subjected to monotonic and cyclic loading. However, the FE model used for the simulations under cyclic loading did include geometrical imperfections which should influence considerably the rotation capacity. The empirical equations provided by the authors included only web and flange slenderness as regression indicators, ignoring the influence of different material steel grades, lateral slenderness, and member length. El Jisr *et al.*, [13] proposed empirical equations to estimate the plastic rotations for steel non-composite beams which were derived from limited tests on European

profiles and the experimental database collected by Lignos and Krawinkler [15] and Hartloper and Lignos [11] which was used for the preparation of American standards in NIST2017 [16]. This database includes a large number of American deep beams with section height to flange width ratio $\frac{h}{b} > 2$ and $0.53m \leq h \leq 1m$, tested in the SAC Joint Venture research project [17]. However, in European steel construction practice, the common beam size is very small compared to U.S (Mele [18]). Large sections typically used in the U.S achieve very low level of plastic rotations (Roeder [19], FEMA355d [20]), this might influence considerably the regression results of the rotations for European beams. Due to the typical redundant spaced frames used in European practice, shallow beams that have higher moment capacity (HEA200-600 with $\frac{h}{b} < 2$) can be adopted when architectural restrictions on ceiling height are present. The buckling behavior and its effect on the rotation capacity is different between shallow and deep beams (Okazaki *et al.*, [21]). Therefore, it is important to consider the behavior of shallow beams for European practice.

9.1.3 Objective

The main objective of this study is to provide improved estimations of the rotation capacity limits of steel beams with European cross section range accounting for several influencing aspects such as the effects of member length, geometrical imperfections, lateral slenderness, and material steel grades; and to provide a clear view on the different buckling behaviors of deep and shallow beams and their effect of the rotation capacity. For this purpose, a robust finite element model validated with experimental data was developed to simulate the behavior of isolated steel members subjected to flexural cyclic loading. A parametric study was conducted on steel beams subjected to cyclic loading considering different influencing parameters. Based on the results obtained in the numerical study and the experimental data of EqualJoints research project (D'Aniello *et al.* [22], Tartaglia *et al.* [23], Landolfo *et al.* [24], Stratan *et al.* [25]), empirical equations were derived to estimate the cyclic rotation capacity limits of steel shallow and deep beams for the revision of EC8-3.

9.2 Code based requirements and performance parameters

EC8-3 considers three performance limit states, namely: Damage limitation (DL), Significant Damage (SD), and Near Collapse (NC). For steel beams, each damage state is quantified in terms of ductility ratio $\mu = \frac{\theta_p}{\theta_y}$, where “ θ_p ” is the beam plastic rotation determined by subtracting the chord rotation at yielding “ θ_y ” from the total chord rotation “ θ ”. For instance, plastic rotations of $1\theta_y$, $6\theta_y$, and $8\theta_y$ are provided by the code for ductility class 1 beams to correspond to DL, SD, and NC, respectively. EC8-3 requires that at DL limit state: only little material plastic deformations are allowed; at SD limit state: beams are supposed to develop their full plastic moment without geometrical instabilities; and at NC limit state: the member can experience geometrical instabilities such as local buckling. These requirements suggest that they were retrieved from monotonic behavior, where a beam could reach large plastic deformations before the initiation of geometrical instabilities. Under cyclic loading, local buckling can initiate at relatively low level of plastic deformation which reduces the interval between DL and SD significantly. ASCE41-06 [26] allows local buckling of beam elements and severe joint distortion at the same level of plastic rotations of SD limit state of EC8-3. Several experimental studies reported that flange buckling can occur at very early stages without causing loss of bending strength (e.g. see Zieman [27] p.832). In such cases, it is very difficult to identify the local buckling initiation from moment rotation hysteresis curves. A confusion can raise between DL and SD limits. Therefore, the rotation at DL is beyond the scope of this study. Based on the definitions commonly adopted in the literature (ref. [4], [13]), the quantification of the rotation capacity limit at SD is considered as the plastic rotation at maximum moment ($\theta_{p\ max}$) obtained from the hysteresis curve, and the rotation at NC ($\theta_{P\ 80\%}$) is considered when the bending strength drop to 80% of the maximum moment in the negative slope of the first cycle envelope. Figure 9-2 illustrates the performance limits considered in this study, jointly plotted with the cyclic moment rotation curve, the first cycle envelope, and the idealized trilinear curve. After the NC limit, the cyclic degradation continues to occur until full collapse of the beam. The rotation at collapse is useful for collapse analysis, however, the beam will experience large level of

plastic deformation accompanied with ultra-low cycle fatigue in the beam or crack in the connection components, which are not considered in the numerical model. Therefore, a precise estimation of the rotation at collapse is beyond the scope of this study. An extrapolation of $\theta_{P\ 80\%}$ to zero bending moment might be a conservative option since the stabilization of the hysteresis curves at large deformation is not included. The plastic rotations $\theta_{p\ max}$ and $\theta_{P\ 80\%}$ are determined using equation (1) and equation (2). Where θ_y is the rotation at yielding (i.e. $\theta_y = \frac{M_y L}{3EI}$). M_y is the bending moment at yielding, L is the length of the cantilever, E is the young modulus, and I is the moment inertia.

$$\theta_{p\ max} = \theta_{max} - \theta_y \quad (19)$$

$$\theta_{p\ 80\%} = \theta_{80\%} - \theta_y \quad (20)$$

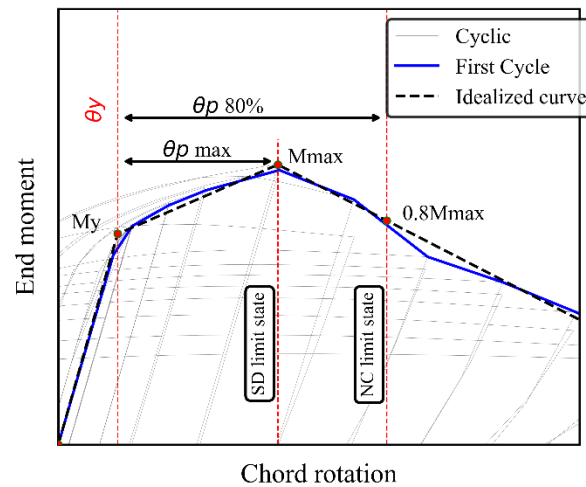


Figure 9-2: Definition of the damage states according to the plastic rotation limits.

9.3 Development of the F.E model and validation

A 3D continuum finite element model was developed in the commercial software ABAQUS® [28]. The model consists of a cantilever that is assumed to reproduce the behavior of the beam from one end to the point of inflection which is assumed at the mid-length of the beam. The finite element model used in this study is shown in Figure 9-3. One edge was fully restrained in the 6 degrees of freedom to ensure the fixed

boundary conditions. The cyclic loading was applied on the free edge of the beam, which is restrained in the X-X direction to avoid out-of-plane displacements, as in the experimental setup of D'Aniello et al [2]. The cyclic loading has been applied as displacement with amplitude history in the strong axis of the beam (y-y axis) at reference point RP-2. Both cross-section edges were constrained as a rigid body to reference points RP-1 and RP-2, to avoid stress concentration. The beam's plates were modelled using quadratic shell elements S4R, known to capture well large inelastic deformation and post-buckling behavior. After a mesh sensitivity, a very refined mesh size was adopted equal to $b/24$, where b is the flange width. The mesh density was kept constant along the full length of the member in order to accurately simulate any eventual lateral and torsional buckling (LTB). Lateral restraints were applied in the flanges to prevent excessive LTB. The unbraced length (L_b), which is the distance between the lateral restraints, was estimated according to ANSI/AISC 341-16 [29] using equation (21). An additional set of beams with $\frac{1}{2} L_b$ were included in the analyses for beams that experienced considerable LTB.

$$LB = 0.095 iz \left(\frac{E}{F_y} \right) \quad (21)$$

Where iz is the radius of gyration of the cross-section, E is the young modulus, F_y is the yield stress obtained from coupon test. Flange and web imperfections are included in the model considering the geometrical manufacturer shape tolerances according to EN 10034 [30], more information are presented later in this paper.

The combined nonlinear isotropic/kinematic hardening plasticity model based on the Chaboche proposal [31] available in the ABAQUS materials library was adopted to capture the complex cyclic behavior of the material. The mathematical expression of the nonlinear isotropic component is presented in equation (22). It captures the cyclic hardening/and or softening behavior of the material through the enlargement or reduction of the Mises yield surface.

$$s^0 = s|_0 + Q_\infty(1 - e^{-be^{pl}}) \quad (22)$$

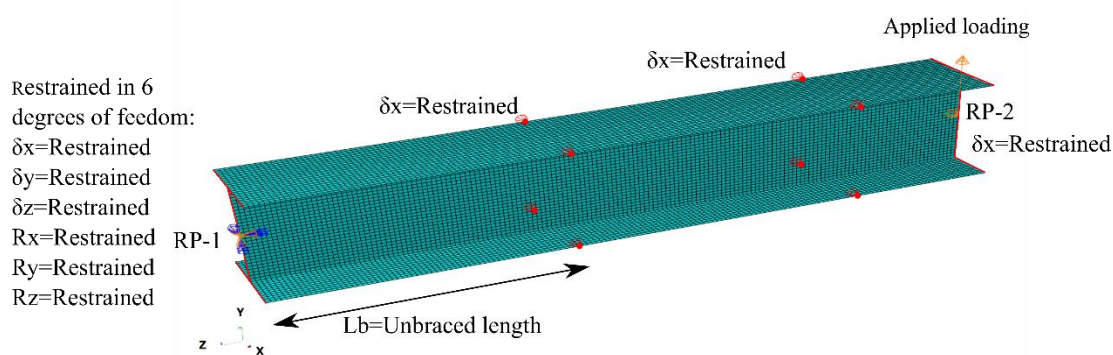


Figure 9-3: Finite element model.

The nonlinear kinematic hardening component describes the translation of the yield surface in stress space through the vector backstress α , to capture complex phenomena such as the Bauschinger effect. The backstress evolution law is the vector function that translates the center of the Mises yield surface. Its mathematical expression is presented in equations (23).

$$\dot{\alpha}_k = C_k \frac{1}{s_0} (s - \alpha) \dot{\epsilon}^{pl} - g_k a_k \dot{\epsilon}^{pl} \quad (23)$$

The cyclic plasticity model was calibrated using cyclic stress-strain data of tests under ascending and alternate protocols tested by Chen et al [32]. The parameters of the material model for S235 and S355 steel grades are presented in Table 9-1.

Table 9-1: Nonlinear Isotropic and Kinematic hardening parameters.

Material	σ_0 (N/mm ²)	Isotropic		Kinematic component	
		Q_∞ (N/mm ²)	b	C (N/mm ²)	γ
S355	370	70	2	12640	91
S235	269	60	4	11086	99

The physical tests conducted by D’Aniello., *et al* [2] (IPE300 and HEA160) and McRare [33] (specimen C0) were simulated in ABAQUS to validate the finite element model developed in this study. The hardening parameters of the material steel grade S235 were used in the model since it has the closest yield strength to the material of each tested specimens. Figure 9-4 illustrates the comparison between the moment-chord rotation cyclic curves obtained from the numerical model and the experimental results. Overall good matching between the numerical and experimental results.

Figure 9-5 compares the failure modes observed in the experiments and in the numerical simulation. As it can be seen the local buckling modes were well captured by the numerical model, where IPE300 beam experienced lateral and torsional buckling, and HEA section experienced anti-symmetric flange local buckling (FLB) and web local buckling (WLB). Specimen C0 also experienced anti-symmetric FLB and WLB.

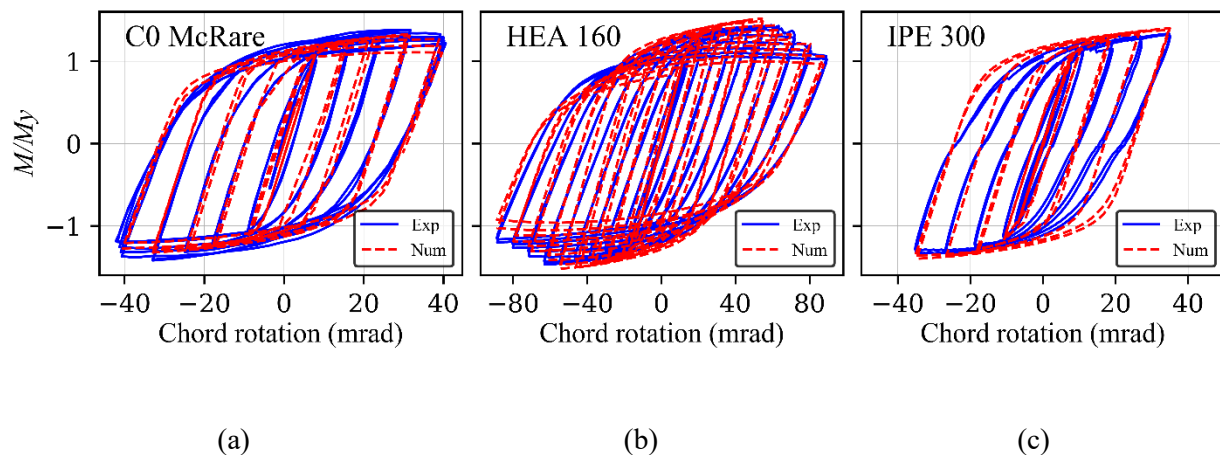


Figure 9-4: Comparison between experimental and numerical results: (a) HEA160 specimen (D’Aniello et al [2]); (b) Specimen C0 (McRae [33]); (c) specimen (D’Aniello et al [2]).

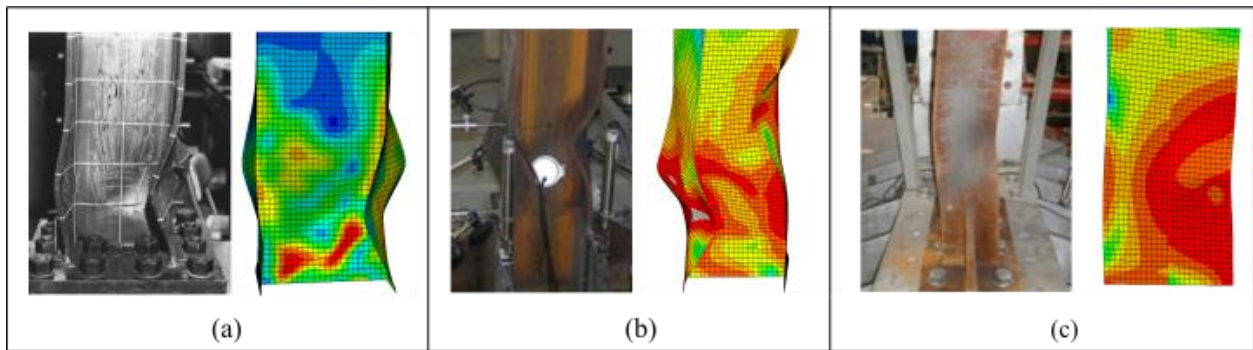


Figure 9-5: Comparison of buckling mode between experimental tests and numerical simulations: (a) anti-symmetric local buckling Specimens C0 (McRae [33]); (b) non-symmetric local buckling of HEA160 (D'Aniello et al [2]); (c) lateral buckling of IPE 300 (D'Aniello et al [2]).

9.4 Numerical investigation

A total of 294 cyclic nonlinear analyses were performed in ABAQUS® software [28] using continuum finite element model that captures well buckling and post-buckling behavior of steel beams subjected to cyclic loading. The simulation program included:

- Deep ($2 \leq \frac{h}{b_f} < 3$) and shallow ($1 \leq \frac{h}{b_f} < 2$) beams with European profiles of cross section range IPE200-600 and HEA200-600.
- The effect of the material steel grade was included in the numerical study by simulating the cyclic behavior of two different steel grades (i.e. S235 and S355) using advanced cyclic plasticity model.
- The geometrical imperfections were explicitly modeled using manufacturer's shape tolerances with different amplitudes for both web and flange imperfections.
- The effect of member length on the rotation capacity was also considered by modelling beams with different lengths ($L=2\text{m}$, 3m , 4m , and 6m).
- The symmetric cyclic loading protocol provided in ANSI/AISC341-16 [29], shown in Figure 9-6, was adopted in this study.

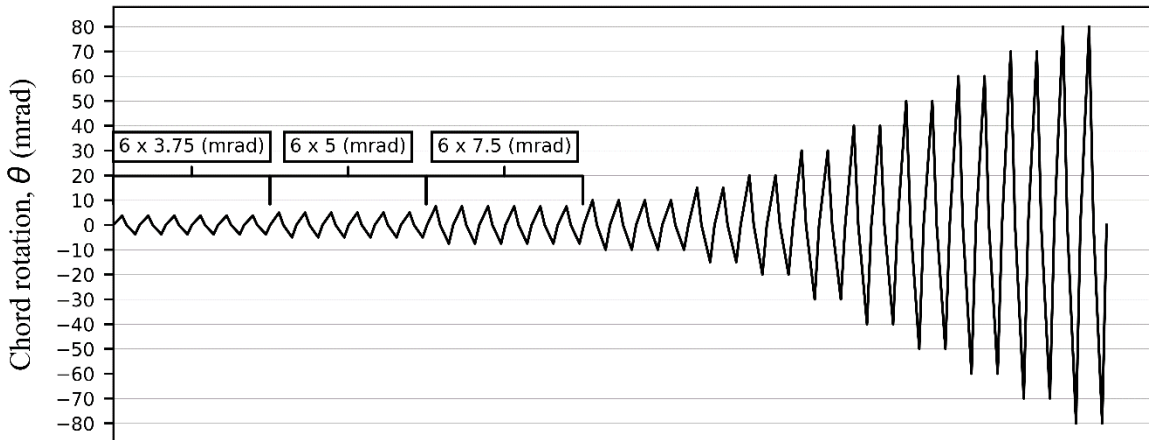


Figure 9-6: Cyclic loading protocol.

9.4.1 Considerations of the yield mechanism and buckling patterns

When fracture is prevented or delayed in MRF connections, geometrical instabilities such as local buckling and LTB control the deformation capacity of seismic moment resisting frames (Okazaki *et al.*, [21]). The extension of the ductile deformation of steel flexural members such as beams lasts when buckling causes significant loss of strength. The loss of bending resistance of a member occurs when the full width of the flange does not remain yielded under compression due to the extensive curvature caused by flange local buckling (FLB) (Kemp [34]). In fact, FLB has long been recognized to be a very important consideration in rotation capacity issues (Galambos [35] p.776). Since flange plates have a non-supported external boundary which makes them more vulnerable to buckle before the web. Nevertheless, the web buckling behavior plays also an important role in the rotation capacity since the web is responsible to provide rotational stiffness to resist FLB. Therefore, the initiation of WLB would allow rapid and more severe development of FLB. In this study, it was observed in all simulations that strength degradation did not occur until WLB initiates.

9.4.1.1 Shallow beams ($HEA\ 1 \leq h/bf < 2$)

Shallow beams experienced mainly local buckling. A typical yielding and buckling pattern of shallow beams observed in the FE analysis is shown Figure 9-8. The beams reached a relatively shorter plastic hinge compared to deep beams. It was observed that FLB could initiate at early stages with very low buckle amplitude that continues to increase slowly until WLB begins. Once WLB was triggered, larger amplitudes were observed for both FLB and WLB in every consecutive imposed loading until reaching the full buckling mechanism. Observations on the FE model and Moment-rotation cyclic curves brought the attention that FLB alone did not cause loss of strength, further inelastic rotation could be reached. However, the instability of flanges has prevented the development of over-strength in moment-rotation curve, as it can be seen in Figure 9-7 (b), where the first cycle curves for HEA beams show a flat plateau after yielding. When, WLB initiated, an immediate drop of bending strength was observed. These observations are in line with the work of Kemp [34] and Decker [36], who tested steel beams and T sections under monotonic loading, and reported that FLB or WLB isolated from each other caused little or no weakening.

9.4.1.2 Deep beams ($IPE\ 2 \leq h/bf < 3$)

A typical local buckling pattern of deep beams laterally restrained according to ANSI/AISC 341-16 [29] is shown

Figure 9-9: Yield mechanism and buckling pattern of typical deep beam (IPE400).

Figure 9-9. The buckling behavior of IPE beams was different than HEA beams. The beams showed longer plastic hinge with material plastic strain hardening for several cycles without experiencing geometrical instabilities. Yielding was also observed at the lateral restraints supports. The buckling mode showed an interaction between of FLB, WLB and LTB near at the plastic hinge region accompanied with a sudden and rapid strength degradation. The backbone curves in Figure 9-7 (a) shows that IPE beams developed significant over strength after yielding until reaching the peak, then a sharp drop of strength can be observed due to the simultaneous development of WLB with FLB and LTB.

Although the beams were laterally restrained, LTB occurred in the unbraced length near the plastic hinge. This behavior has been also observed in experimental tests on restrained beams with IPE profile (D’Aniello *et al.*, [2]). In order to check the influence of LTB on the rotation capacity of deep beams, another set of simulation was conducted with beams having braced length of $\frac{1}{2}$ LB. The results showed that the beams experienced mainly local buckling with larger plastic hinge and rotation capacity.

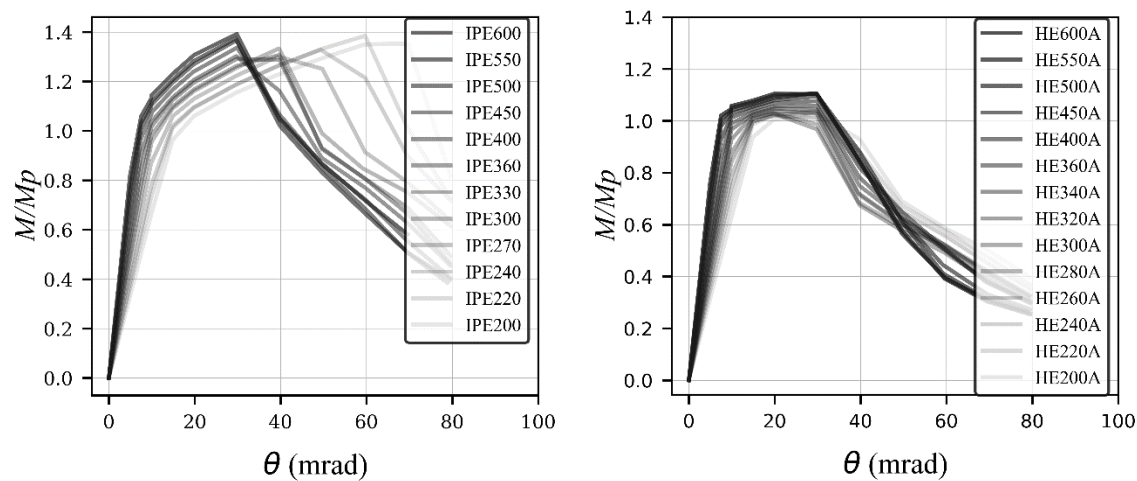
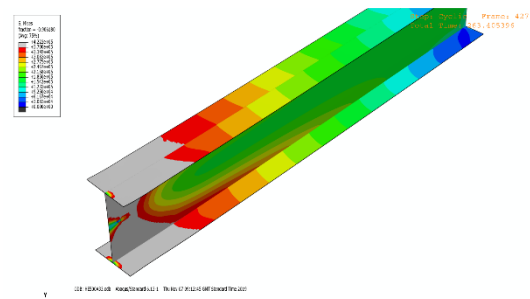
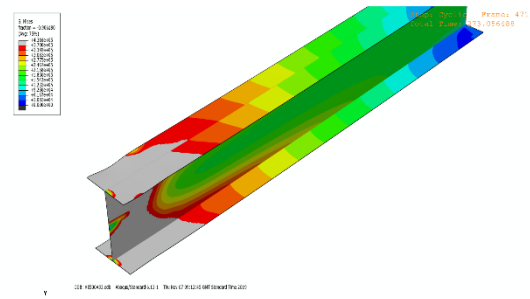


Figure 9-7: Typical first cycle cyclic first cyclic curves for IPE 200-600 and HEA 200-600.

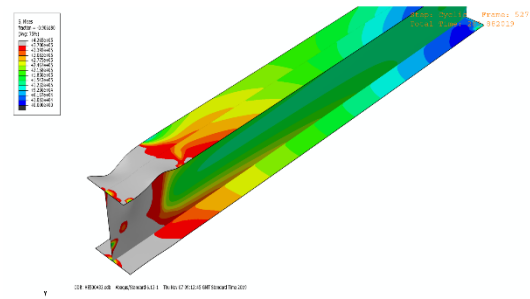
Chapter 9: Deformation capacity of I steel beams subjected to cyclic loading for the seismic design of structures



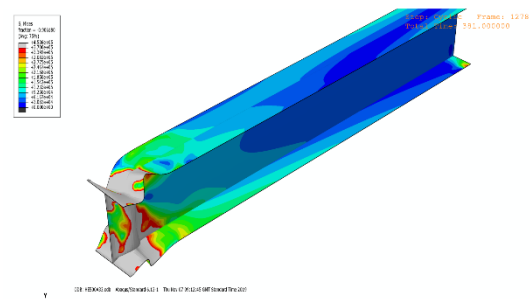
(a) Material plastic yielding



(b) FLB initiation



(c) WLB initiation



(d) Full local buckling

Figure 9-8: Yielding mechanism and local buckling pattern of typical shallow beam HEA400.

Chapter 9: Deformation capacity of I steel beams subjected to cyclic loading for the seismic design of structures

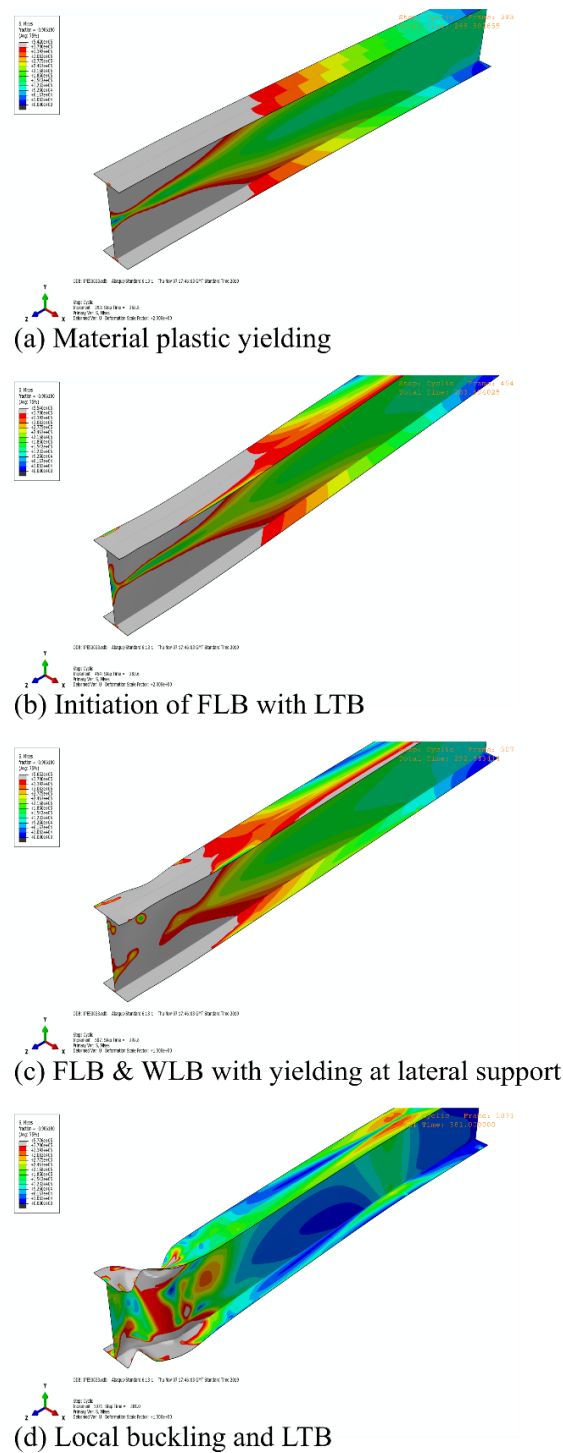


Figure 9-9: Yield mechanism and buckling pattern of typical deep beam (IPE400).

9.5 Parametric study results and discussion

9.5.1 Influence of geometrical imperfections

This section presents a parametric study on the effect of geometrical imperfection. The main goal of this study is to show the importance of considering geometrical imperfections in the cyclic analysis contrarily to Araujo *et al.*, [4] assumptions. In this study, the geometrical imperfections were explicitly modelled in the numerical model based on tolerances on shape and dimensions allowed by the European Standard for dimensions of I and H beams EN10034 [30] and considering the approach used in the literature (Elkady and Lignos [37]). As shown in Figure 9-10, flange imperfections were considered using symmetric and anti-symmetric out of squariness shape deviation from the nominal section. Web imperfection was modelled with bow shape following the web buckling mode shape. The combination of imperfection types and their respective amplitudes considered in this parametric study are presented in Table 9-2. The amplitudes considered for flange imperfections are 2%, 0.4%, and 0.2% ; and for web imperfections are 0.4%, and 0.2%. The value of 2% is maximum tolerance permitted by EN10034 [30] for section having flange width $b \geq 110\text{mm}$, 0.4% and 0.2% were considered by engineering judgement and recommendation from the literature [37].

Figure 9-11 illustrates the variation of $\theta_{P80\%}$ for IPE and HEA beams considering the imperfections presented in Table 9-2. As can be seen, perfect beams achieve much higher plastic rotations than imperfect ones for both IPE and HEA beams. This is an evidence of the importance of considering geometrical imperfections. Symmetric and anti-symmetric flange imperfections showed symmetric and anti-symmetric buckling mode shapes. The shape of the flange imperfections did not show difference on the rotation capacity. The imperfection FS2_W04, which is a combination of flange imperfection with amplitude equal to the maximum tolerance allowed by the code and web imperfection with amplitude similar to ref. [37], was considered for all other studies presented in this paper.

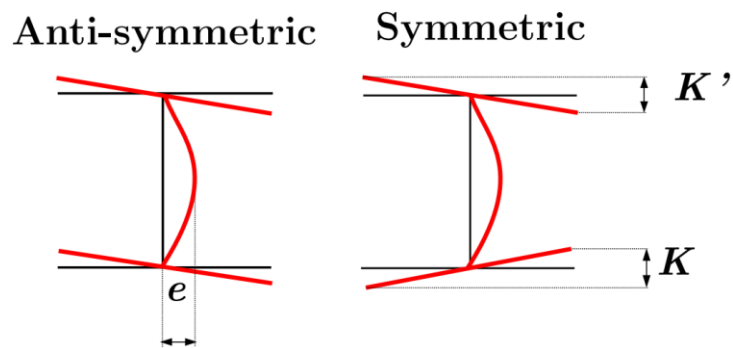


Figure 9-10: geometrical imperfections.

Table 9-2: Imperfection details.

Imperfection	Imperfection type	$K+K'$	e
NO_IMP	No	--	--
FS2	Flange (sym)	2% b	--
FS04	Flange (sym)	0.4% b	--
FS2	Flange (anti-sym)	2% b	--
FAS04	Flange (anti-sym)	0.4% b	--
W04	Web only	--	0.4% h
W02	Web only	--	0.2% h
FS2_W04	Flange (sym) -web	2% b	0.4% h

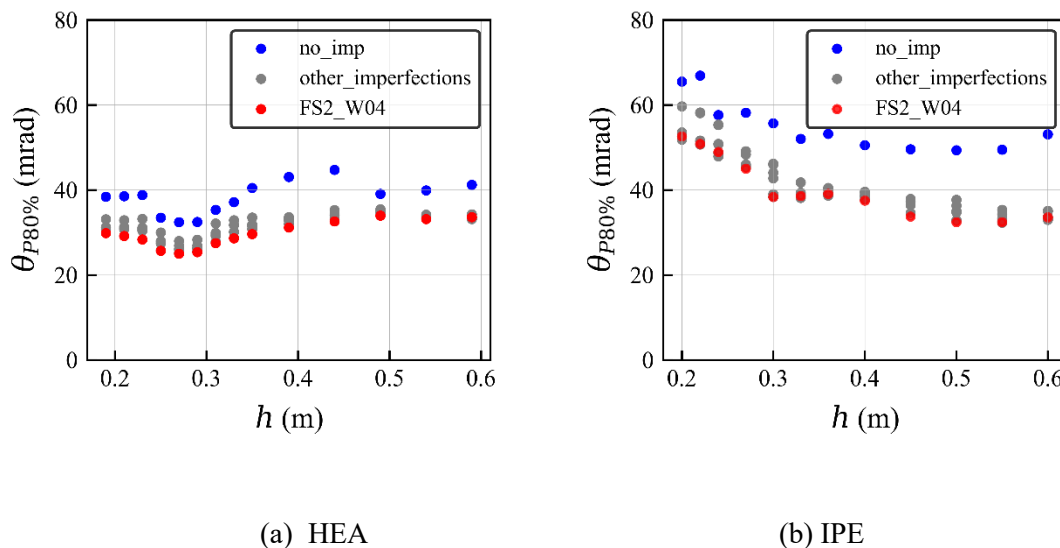


Figure 9-11: Effect of imperfections on $\theta_{P80\%}$ for: (a) HEA beams; (b) IPE beams.

9.5.2 Influence of material steel grade

It is widely recognized that the material ductility influences the inelastic deformation capacity of a steel member, as explained in Appendix 1 AISC360 (2016) [29]. An early study conducted by Takanashi [38] on the cyclic behavior of steel beams showed that specimens that had higher yield stress achieved lower rotation capacity. However, due to the lack of experimental data and in practice usually one steel grade dominates the construction market, a quantification of this effect is not explicitly provided in code provisions. In Europe, S235 steel dominated the steel construction market since the beginning of the 20th century [39], however it is less commonly used nowadays. Therefore, it is important to investigate the effect of the different steel grades on the rotation capacity which can be useful for the assessment of new and old existing buildings.

The results presented in this section compares the plastic rotations of IPE and HEA beams with two steel grades, namely, S235 and S355. Where a higher material ductility of S235 steel was observed in the results of coupon tests (Chen *et al.*, [32]). As can be seen in Figure 9-12, HEA beams with S235 show a

considerable higher $\theta_{P80\%}$ compared to the beams with S355. However, for IPE beams the comparison shows no significant difference. One possible explanation for this little influence of material ductility on IPE beams can be related to the rapid development of WLB, which was the main cause of strength degradation when combined with LTB or FLB. As stated previously, IPE beams experienced WLB simultaneously with other buckling modes. So, once the buckling was triggered, abrupt strength degradation began in the numerical model. Thus, no additional ductility due to material hardening could be developed.

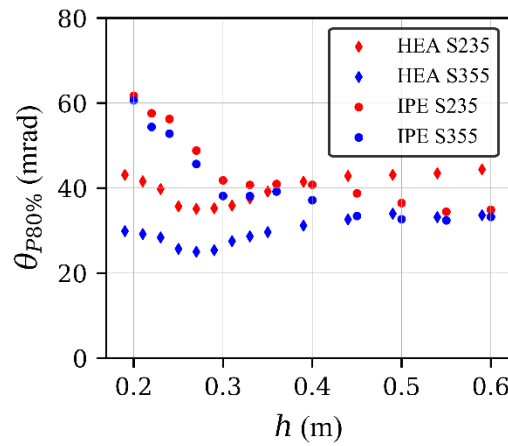


Figure 9-12: Effect of material steel grade on the rotation capacity on HEA and IPE beams of 2m.

9.5.3 Influence of width to thickness ratios ($b/2t_f$ and h_w/t_w)

The importance of the influence of $b/2t_f$ and h_w/t_w on the rotation capacity was recognized since the early experimental studies on the behavior of steel I-shaped beams (Kuhlmann [40], Johnson [41]). Note b is the flange width, h_w is the web height as defined in Eurocode 3 [1], t_f is the flange thickness, and t_w is the web thickness. Figure 9-13 illustrates the variation of θ_{pmax} and $\theta_{p80\%}$ versus $b/2t_f$ and h_w/t_w for IPE and HEA beams. For HEA beams, Figure 9-13 (a) and (b) depict clearly that the plastic rotations are strongly dependent on $b/2t_f$, which is consistent with the local buckling patterns shown previously in this

paper, where FLB initiated first. HEA profiles cover a very large interval of $b/2t_f$, ranging from very slender ($b/2t_f = 10.8$) to very stocky ($b/2t_f = 6.0$). This helps to show the clear relationship between $b/2t_f$ and the plastic rotations since intuitively very slender flanges reach lower rotation capacity and vice versa. On the other hand, the rotations of IPE beams shown in Figure 9-13 (c) and (d) reveal a clear dependence on h_w/t_w . The influence of $b/2t_f$ on IPE beams is not clear since the $b/2t_f$ varies in a short interval and the data show lot of scatter.

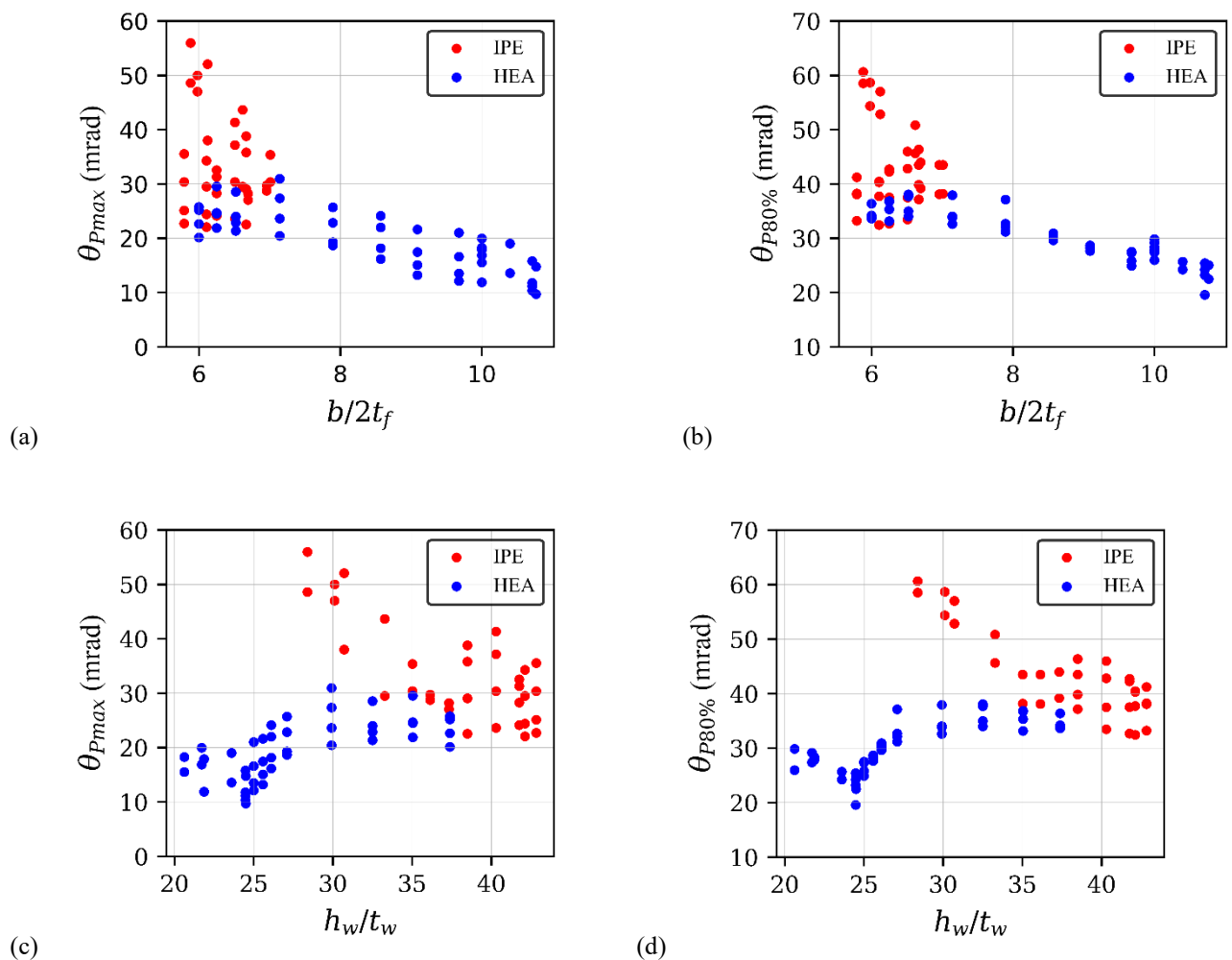


Figure 9-13: Variation of θ_{pmax} and $\theta_{p80\%}$ vs c/t_w and $b/2t_f$ for IPE and HEA beams.

9.5.4 Influence of member length

The influence of member length on the rotation capacity can be a subject of controversy. Plastic rotations (θ_{pmax} and $\theta_{p80\%}$) and their corresponding rotational ductility ratio (μ_{max} and $\mu_{80\%}$) adopted by assessment codes provide a paradoxical description on the effect of member length. This can be explained in Figure 9-14 (a) and (b), where the hysteresis loops of two IPE500 beams of $L=2$ and $L=4m$ are plotted, respectively. The beam with $L = 4m$ reaches higher θ_{pmax} than the beam with $L = 2m$, with values of $\theta_{pmax} = 28.3 \text{ mrad}$ and $\theta_{pmax} = 24.1 \text{ mrad}$, respectively. However, due to the high influence of the length on the yield rotation $\theta_y = \frac{M_y L}{3EI}$, the ductility ratio will decrease significantly. Longer beams achieve higher values of yield rotation with larger proportions of increase compared to plastic rotations. For example, in this case, when moving from the beam of $L = 2m$ to $4m$, the plastic rotations increase by a ratio of 1.17, but the yield rotation increase with a higher ratio equals to 2. So, when θ_{pmax} is normalized to θ_y to obtain the ductility ratio μ_{max} , it results with a decrease of ductility ratio by a factor $\frac{1.18}{2} = 0.59$. Anastasiadis *et al.*, [42] proposed a new classification of steel beams with European profiles based on ductility ratio, and downgraded the classification of longer beams. Thus, despite the higher or approximately similar plastic deformations that can be reached by longer beams, the ductility ratio will be always more penalized by the increase of the yield rotation. This raises the concern of whether constant ductility ratio limits as prescribed in the codes give a fair measure of ductility or damage state of the beams. Lignos and Krawinkler [15] also noticed that for beams section height greater than 533mm, and stated that the ductility ratio provides a misleading description of the beam deformation capacity.

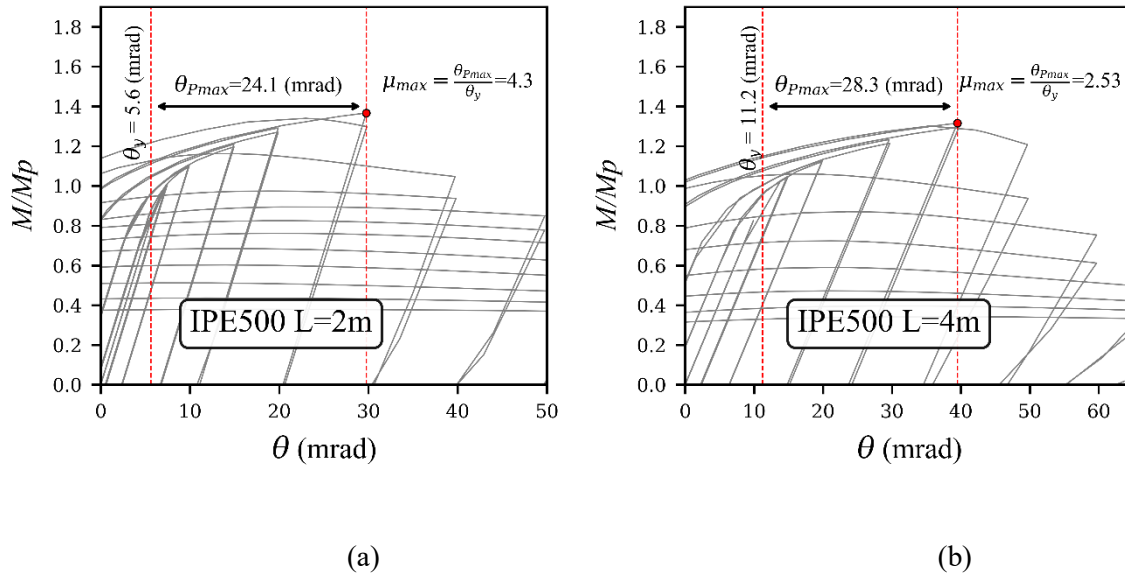


Figure 9-14: Cyclic hysteresis loops of IPE500 beams with two different beam lengths: (a) $L=2\text{m}$; (b) $L=4\text{m}$.

The results of the parametric numerical study are presented in Figure 9-15, which shows the plastic rotations of IPE and HEA beams with $L=2,3,4$, and 6m . For IPE beams, Figure 9-15 (a) and Figure 9-15 (b) show clearly that the θ_{max} and $\theta_{80\%}$ increase considerably with member length. The dependence of the plastic rotations of IPE beams on the ratio h/L is shown in Figure 9-17, where the rotations are increasing with h/L , which will be considered as a regression variable in the empirical equations. Figure 9-15 (c) and Figure 9-15 (d) depict scattering of the plastic rotations of HEA beams with no clear indication on the dependence on L , except in Figure (3-b) for beams with $0.39 \leq h \leq 0.6$ (i.e. HEA400-600) that show a slight tendency of increase proportionally with L that can be assumed negligible. Therefore, the effect of member length will be not considered in the regression analysis for HEA beams.

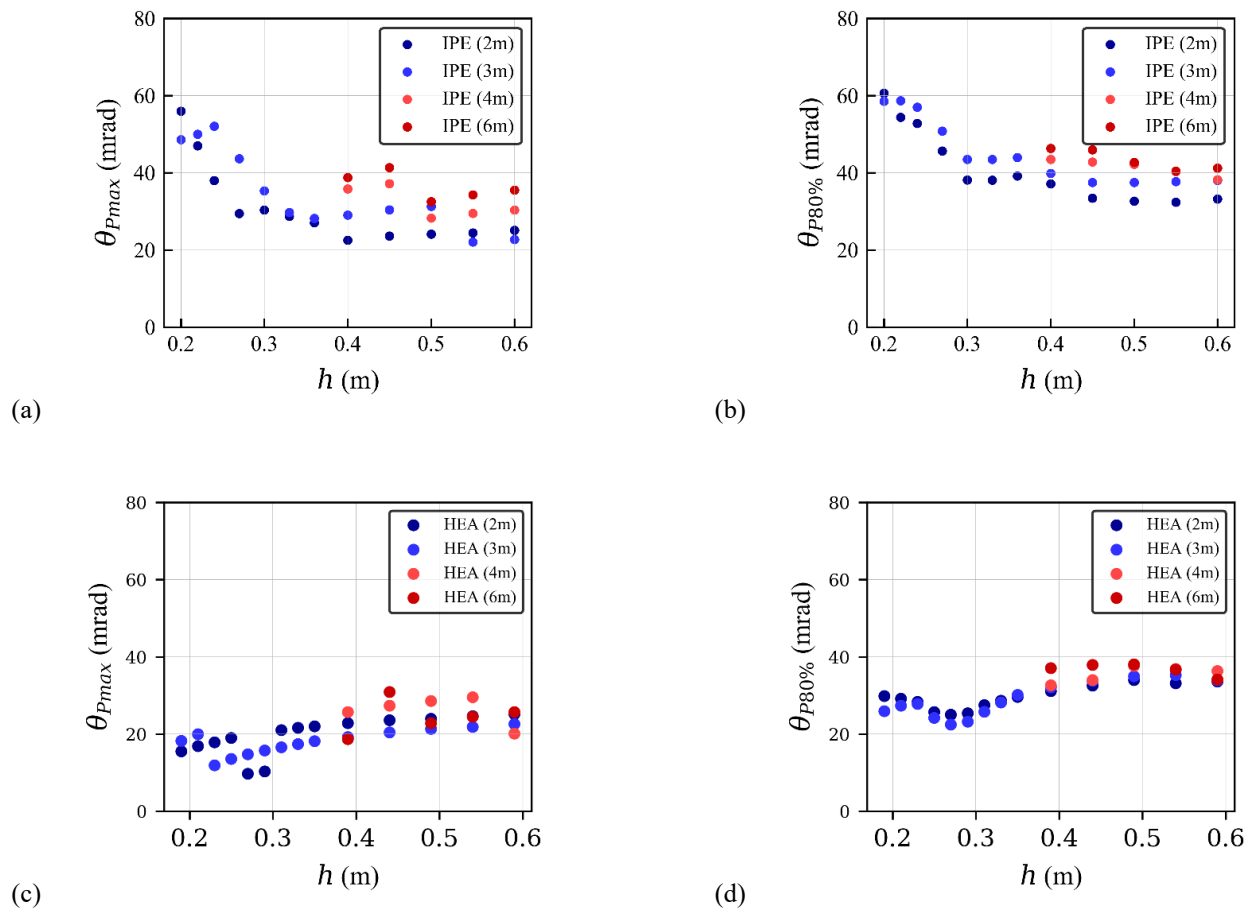


Figure 9-15: Effect of member length on IPE and HEA beams.

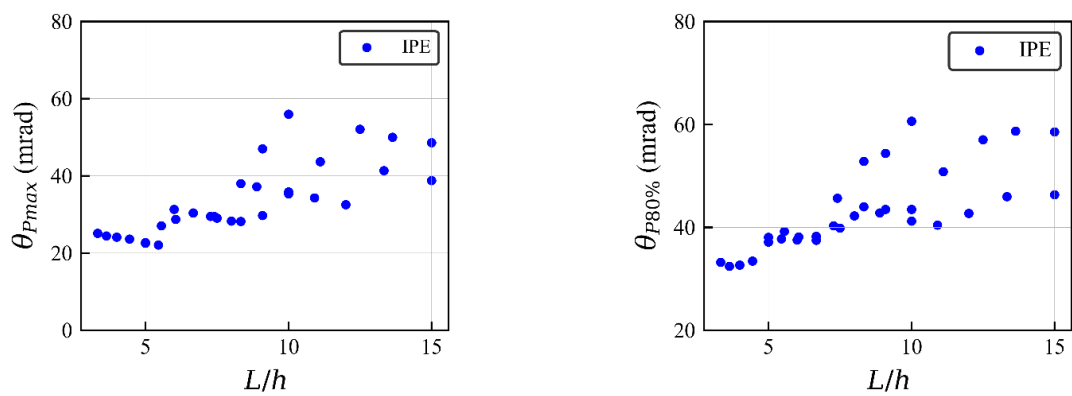


Figure 9-16: Dependence of the plastic rotations on L/h ratio.

9.6 Regression analysis and validation

Based on the numerical results of the parametric study and the experimental results collected from the literature ([2], [24]), empirical equations are derived to estimate the plastic rotation of θ_{pmax} and $\theta_{p80\%}$ for IPE and HEA steel beams. The input parameters included in the multiple regression analysis are the ones that were found to have a considerable influence in the parametric study.

9.6.1 Experimental results collection

The experimental results were collected mainly from EqualJoints research project (Landolfo *et al.*, [24]) and one specimen from D'Aniello *et al.* [2] (i.e. IPE300). The experimental results from EqualJoints were collected from beams tested as part of full-strength rigid connections, which exhibited plastic deformations exclusively in the beams. The contribution of the panel zone and the other connection components to the nonlinear response was reported negligible. The important details of the experimental tests and results related to this study are summarized in Table 9-3.

Table 9-3: Experimental results of full strength and rigid connection and steel isolated beam (Landolfo et al., [24] and D'Aniello et al. [2]).

Index	Specimen	Beam	Fy (MPa)	L (m)	Θ_y (mrad)	Θ_p max (mrad)	Θ_p 80 (mrad)
1	IPE 300	IPE300	300	1.78	8.1	26.9	Nan
2	EH1-TS35-C1	IPE360	400	2.12	14.9	27.7	44.7
3	EH1-TS35-C2	IPE360	400	2.12	16.1	28.1	50.7
4	EH1-TS45-C1	IPE360	400	2.12	15.1	27.6	45.2
5	EH1-TS45-C2	IPE360	400	2.12	14.4	23.9	40.6
6	EH2-TS35-C1	IPE450	400	2.12	12.7	24.4	40.5
7	EH2-TS35-C2	IPE450	400	2.12	12.2	25.5	42.3
8	EH2-TS45-C1	IPE450	400	2.12	12.1	25.0	38.5
9	EH2-TS45-C2	IPE450	400	2.12	12.3	23.8	43.0
10	EH3-TS35-C1	IPE600	400	2.12	11.5	20.4	Nan
11	EH3-TS35-C2	IPE600	400	2.12	12.1	15.0	39.0
12	EH3-TS45-C1	IPE600	400	2.12	9.3	19.3	37.3
13	EH3-TS45-C2	IPE600	400	2.12	9.6	19.1	37.5
14	ES1-TS-F-C1	IPE360	400	3	15.6	31.3	51.6
15	ES1-TS-F-C2	IPE360	400	3	16.4	30.8	46.0
16	ES2-TS-F-C1	IPE450	400	3	11.8	36.4	50.7
17	ES2-TS-F-C2	IPE450	400	3	15.5	32.4	46.4
18	ES3-TS-F-C1	IPE600	400	3	7.8	20.0	40.7
19	ES3-TS-F-C2	IPE600	400	3	10.2	17.7	Nan

9.6.2 Regression analysis for IPE beams

For IPE beams, the parameters included are the width to thickness ratio of the cross section, member length, material steel grade, and lateral slenderness. Although the flange slenderness $b/2t_f$ did not show a clear influence on rotations, it was included in the regression analysis to account for the interaction between

flange and web slenderness. The proposed equations for IPE beams are presented in Equation (6) and (7).

The graphical fitting test of the equations is presented in Figure 9-17

$$\theta_{Pmax} (rad) = 1.235 \left(\frac{b}{2t_f} \right)^{-0.146} \left(\frac{h_w}{t_w} \right)^{-0.938} \left(\frac{L}{h} \right)^{0.321} \left(\frac{L_b}{iz} \right)^{-0.188} \left(\frac{F_y}{355} \right)^{-0.658} \quad (24)$$

$$\theta_{80\%} (rad) = 1.767 \left(\frac{b}{2t_f} \right)^{-0.228} \left(\frac{h_w}{t_w} \right)^{-0.83} \left(\frac{L}{h} \right)^{0.0965} \left(\frac{L_b}{iz} \right)^{-0.137} \left(\frac{F_y}{355} \right)^{-0.226} \quad (25)$$

Where : h is the section height, b is the flange width, t_w is the web thickness, h_w is the web depth as defined in Eurocode 3, t_f is the flange thickness, L is the length of the cantilever (note: in framed structures, the half shear span should be taken), L_b is the lateral unbraced length, and f_y is the expected yield stress in N/mm^2 .

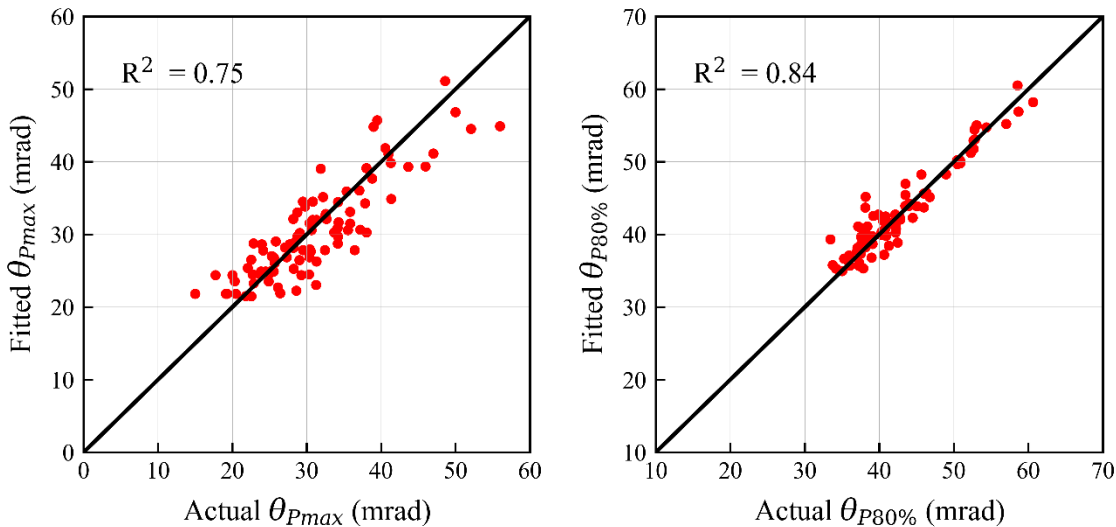


Figure 9-17: Fitting test of equations (6) and (7) for IPE beams.

9.6.3 Regression analysis for HEA beams

For HEA beams, L_b/iz and L/h were excluded from the empirical equations since they did not show considerable influence on the rotations. This is in line with the observations of Okazaki *et al.*, [21] who showed that the rotations of shallow beams were insensitive to L_b/iz for beams that are within the same

flange and web slenderness range. Equation (8) and (9) are proposed to estimate the plastic rotations for HEA beams. The graphical fitting test of the equations is presented in Figure 9-18.

$$\theta_{Pmax}(rad) = 4.59 \left(\frac{b}{2t_f} \right)^{-1.48} \left(\frac{h_w}{t_w} \right)^{-0.69} \left(\frac{F_y}{355} \right)^{-1.05} \quad (26)$$

$$\theta_{P80\%}(rad) = 3.94 \left(\frac{b}{2t_f} \right)^{-1.12} \left(\frac{h_w}{t_w} \right)^{-0.75} \left(\frac{F_y}{355} \right)^{-0.87} \quad (27)$$

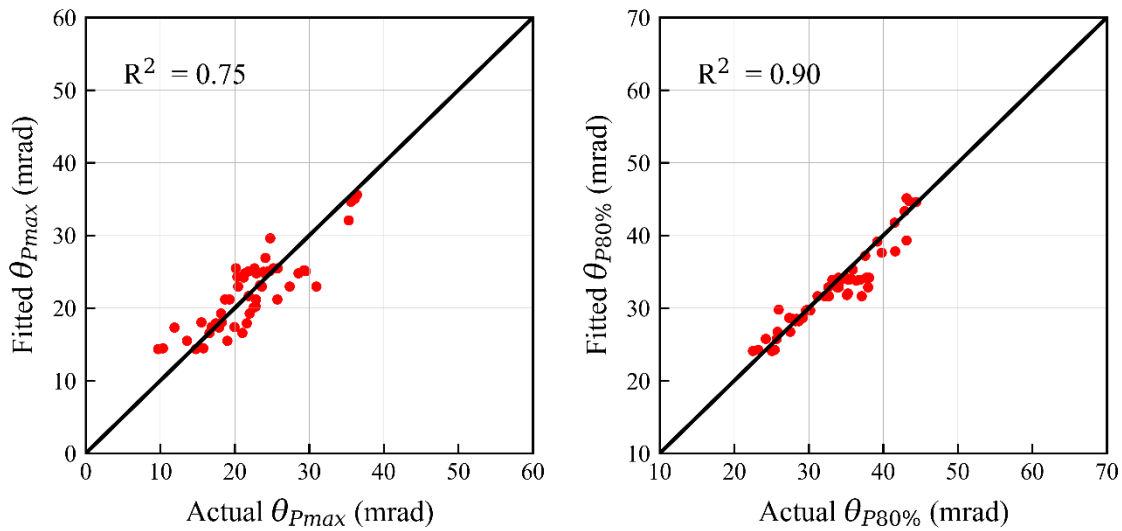


Figure 9-18: Fitting test of the Equation 8 and 9 for HEA beams.

9.6.4 Comparison with proposals in the literature

9.6.4.1 IPE beams

Figure 9-19 plots the variation of θ_{Pmax} for IPE beams estimated using Equation (6) and the different proposals from the literature (i.e. EC8-3, Araujo et al., [4], El Jisr et al., [13] and NIST [16]), assuming $L = 2m$, $\frac{L_b}{I_z} = 20$, and S355 with expected yield stress $F_y = 355 \times 1.2$ MPa as recommended by NIST2017 guidelines[16]. Experimental results of beams with approximately similar properties are also plotted in the figure. As can be seen, the rotations estimated by Equation (6) predict reasonably well the experimental results. The rotations proposed by EC8-3 and Araujo et al [4] are very high compared to the experimental

data. The estimation provided by El-Jisr *et al.*, [13] are very low compared to experimental data. The rotations proposed by NIST [16] and El-Jisr *et al.*, [13] are quite identical particularly for beams with $h > 0.3$. This was expected as the two later proposals were derived almost from the same experimental results. The reason for relatively lower values of θ_{pmax} is attributed to the cross section range used to derive the empirical equations, which included many beams with large sections that have very low θ_{pmax} , as discussed in the beginning of the paper. Thus, the trend of the empirical equation tends to be very low as the section high increases.

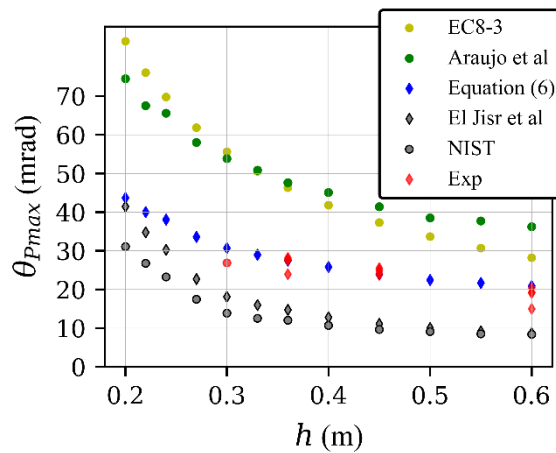


Figure 9-19: Comparison between the plastic rotations at maximum moment θ_{pmax} of IPE beams ($L=2m$ assuming S355 steel and $Lb/iz=20$) proposed in Equations(6), EC8-3, Araujo *et al* [4], EL Jisr *et al.* [13], and NIST2017 [16].

The comparison of the estimated θ_{pmax} with the full experimental results presented in Table 3 is illustrated in Figure 9-20 (a), where $\frac{Lb}{iz} = 20$ was assumed to avoid the influence of LTB as in the experimental results of EqualJoints (Landolfo *et al.*, [24]) and $\frac{Lb}{iz} = 53$ for IPE300 tested by D'Aniello *et al.* [2]. it can be seen the rotations estimated using Equation (6) provide the best fit with a variation within ± 0.5 ratio. Figure 9-20 (b) illustrates the comparison for $\theta_{p80\%}$, Equation (7) provides a good estimate with variation within ± 0.2 ratio. The rotations proposed by El Jisr *et al.* [13] show also a good matching with $\theta_{p80\%}$ of the experimental results. However, due to the considerably low values of θ_{pmax} proposed by El Jisr *et al.*

[13] shown in Figure 9-20 (a), their proposal do not match the degradation trends in the cyclic curves of the experimental results as shown in Figure 9-21 and Figure 9-22. Where, it can be seen that the proposal presented in Equation (6) and (7) reflect reasonably well the behavioural trends observed in the experimental results.

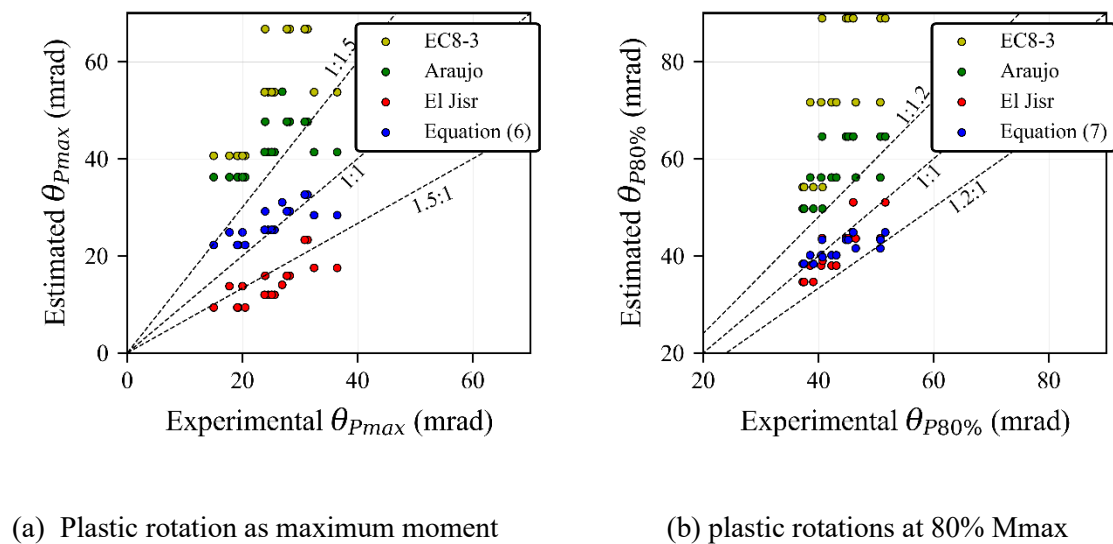


Figure 9-20: comparison between Experimental and estimated plastic rotations.

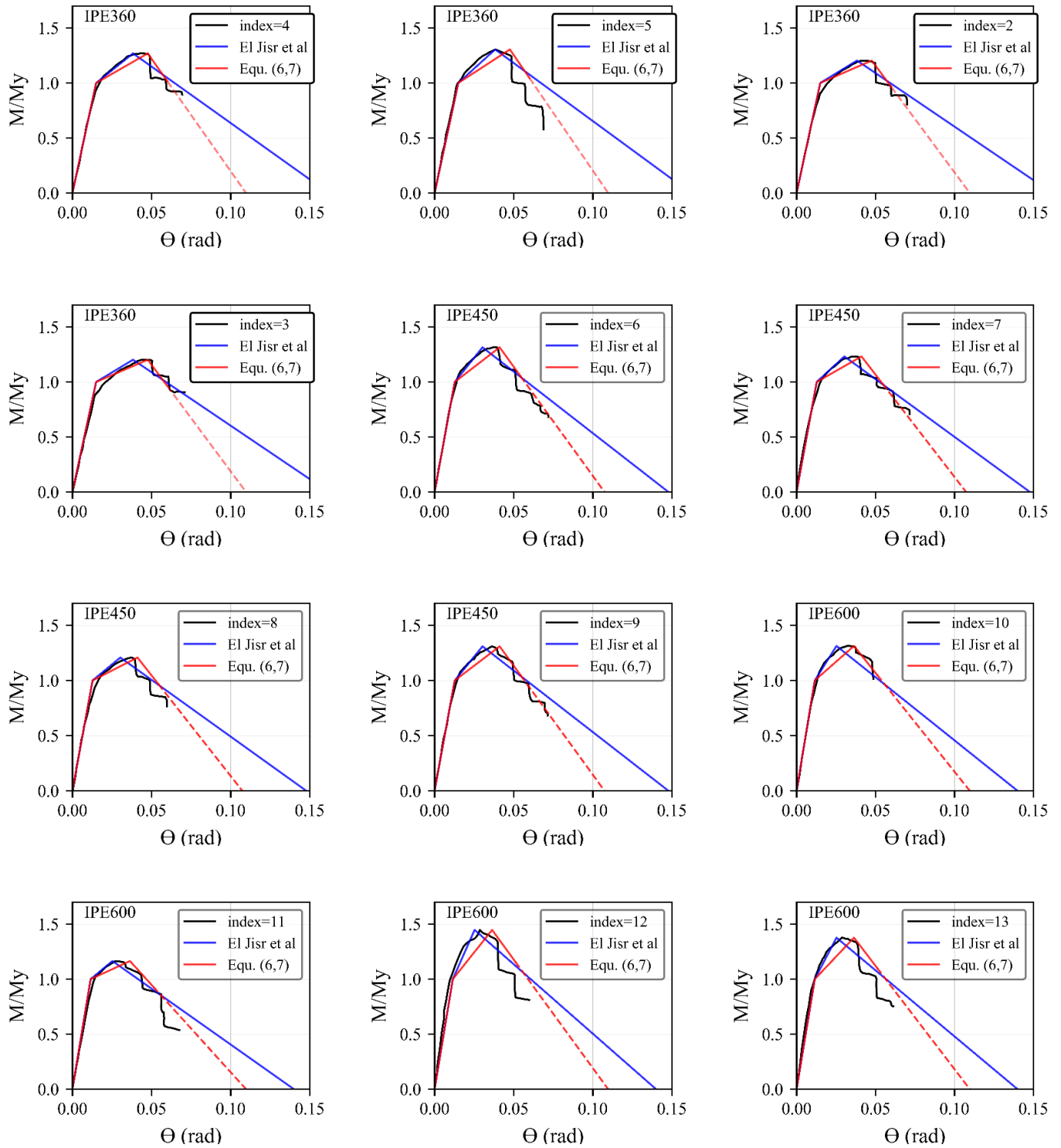


Figure 9-21: Comparison between cyclic backbone curves of experimental results (Stratan et al. [25]) and the first cycle backbone curves proposed in Equ. (7, 8) and the proposal of El Jisr et al. [13].

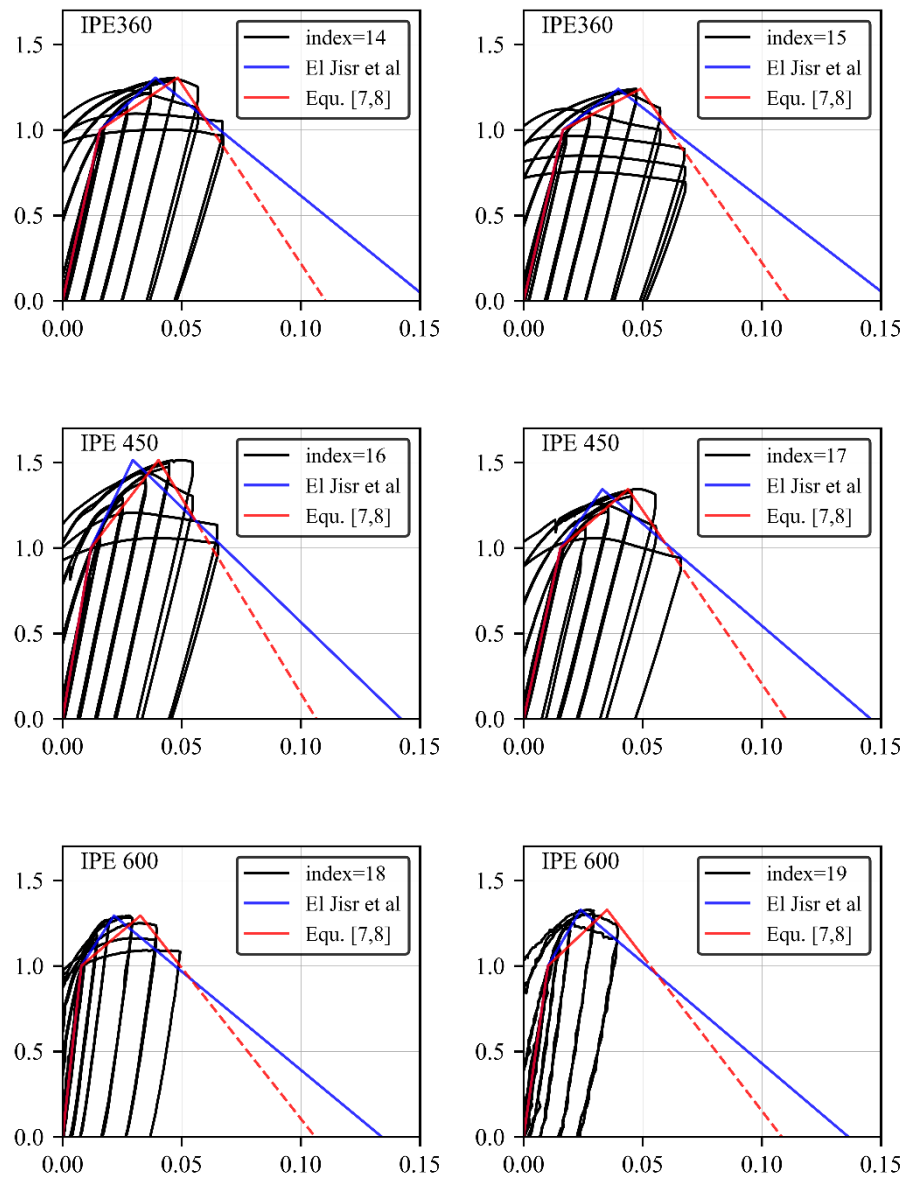
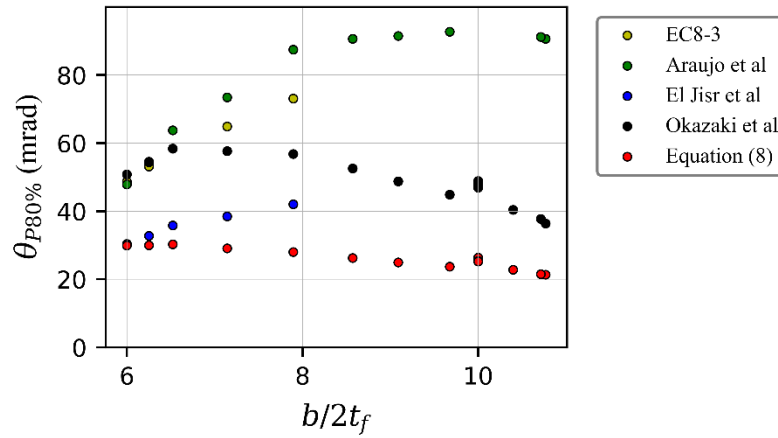


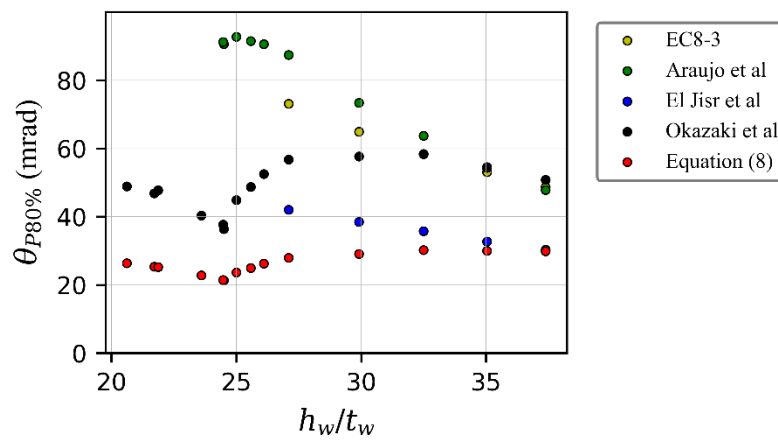
Figure 9-22: Comparison between cyclic experimental results (D'Aniello et al.[22], [23]) and the first cycle backbone curves proposed in Equ. (7, 8) and the proposal of El Jisr et al. [13].

9.6.4.2 HEA beams

To the knowledge of the authors, very limited experimental data on HEA beams is available in the literature. Therefore, the rotations of Equation (9) and (10) are compared with the available rotations proposed in the literature based on the observations on the behavior presented in this numerical study and in the work of Okazaki *et al.*, [21] in which the behavior of shallow beams was investigated. Figure 9-23 illustrates the variation of $\theta_{P80\%}$ versus $b/2t_f$ and h_w/t_w across HEA200-600 beams assuming $L=3m$, $L_b/iz = 20$, and S355 with expected yield stress $F_y = 355 \times 1.2$ MPa. As expected, Figure 9-23 (a) shows that $\theta_{P80\%}$ are inversely proportional to $b/2t_f$ and Figure 9-23 (b) shows no clear dependence of the rotations on h_w/t_w . Similar trends can be observed with the rotations of Okazaki *et al.*, [21]. The significant difference of the level of rotations is due the different cross section range used for the regression and because of the different acceptance criteria where the rotations of Okazaki *et al.*, [21] were obtained when the bending moment drop to $M = M_y$, which can result in significant difference. Besides, a plastic rotation level of $\theta_p > 40mrad$ as estimated by the equation of Okazaki *et al.*, [21] is not reasonable for low ductility beams that have $8 \leq \frac{b}{2t_f} \leq 10$, lower values as estimated by Equation (8) are more justified. Araujo *et al* [4] empirical equation provides very high values for beams with larger $b/2t_f$, which is counterintuitive. The rotations of EC8-3 and El Jisr *et al.* [13] are limited to beams having $6 \leq \frac{b}{2t_f} \leq 8$. The estimation of El Jisr *et al.*, [13] provides also rotations increasing with $b/2t_f$, this is mainly because these rotations were derived from deep beams only, therefore their trend is decreasing with h_w/t_w , as shown in Figure 9-23 (b).



(a)



(b)

Figure 9-23: Comparison of plastic rotations HEA beams proposed in Equations(8) with EC8-3, Araujo et al [4], EL Jisr et al. [13], and Okazaki et al., [21] (Note: the rotations of Okazaki et al are obtained at moment $M=My$).

9.7 Conclusion

This paper focuses on the characterization of the rotation capacity limits of steel deep and shallow beams with European profiles (i.e. IPE $2 \leq h/bf < 3$, and HEA $1 \leq h/bf < 2$) part of seismic moment resisting frames. These rotations can be useful for the improvement of Eurocode 8 part 3 provisions. A continuum finite element model was developed to capture the complex cyclic behavior of the material as well as the buckling and post buckling behavior. This model was validated with past experimental cyclic tests results. A parametric study that includes 294 finite element simulations of beams subjected to cyclic loading was conducted. The effect of different influencing parameters on the rotation capacity were investigated such as the effect of geometrical imperfections, local slenderness, material steel grades, lateral slenderness, and member length. The plastic rotations at maximum moment and at 80% of maximum moment were quantified. Based on the numerical results and some experimental results collected from the literature, empirical equations are derived using multiple regression analysis to estimate the rotation capacity limits of IPE and HEA beams. Finally, comparison with the rotations provided in the related literature is presented. The main findings of this paper can be summarized to:

- The buckling behavior of shallow and deep beams is different, where shallow beams experience mainly local buckling and deep beams a combination of local buckling and lateral torsional buckling.
- It is important to include the effect of the ductility of the different material steel grades particularly for shallow beams where substantial increase of rotation capacity was observed.
- The rotation capacity of the IPE beams was mainly dependent on h_w/t_w , whereas HEA beams were controlled by $b/2t_f$.
- Although the higher plastic rotations achieved by longer beams, the ductility ratio will decrease due to the strong influence of the member length on the yield rotation. Therefore, the ductility ratio does not provide a fair description of the rotation capacity of steel beams.

- The rotation capacity of IPE beams increased considerably with member length, whereas little influence was observed for HEA beams which can be assumed negligible.
- The empirical equations provided in this paper were compared with the other proposal in the literature. For IPE beams the rotations were thoroughly compared with past experimental data. For HEA beams, due to the lack of experimental data, the comparison was based on theoretical considerations.
- The empirical equations provided in this paper were found to provide the most reasonable estimation of the rotations of IPE and HEA beams.

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**Chapter 10: The application of CFRP to extend the
deformation capacity of steel beams subjected to
cyclic loading**

10 The application of CFRP to extend the deformation capacity of steel beams subjected to cyclic loading

Abstract

This chapter investigates the flexural cyclic behaviour of I-shaped hot rolled steel deep section used as beams in moment-resisting frames (MRF) featuring a carbon fiber reinforced polymer (CFRP) patch on the web through advanced finite element analysis. The main goal of the CFRP reinforcement is to increase the rotation capacity of the member without increasing the overstrength in order to avoid compromising the strong column-weak beam condition in MRF. A reduced finite element model of a steel beam is developed and validated with experimental data. The CFRP patch is modelled considering fracture in the adhesive layer using the cohesive zone modelling (CZM) technique that is able to capture the crack initiation and propagation. Different adhesive types are investigated where the CZM parameters are calibrated from high fidelity fracture mechanics tests that are thoroughly validated in the literature. This includes a rigid adhesive commonly found in the construction industry and two tough adhesives used in the automotive industry. The results revealed that the CFRP patch can increase the rotation capacity of a steel member considerably when using tough adhesives.

10.1 Introduction

Steel moment-resisting frames are commonly adopted in earthquake-prone regions due to their ability to dissipate energy through plastic hinges in pre-defined locations such as beam ends. High ductility class hot-rolled steel profiles (i.e. class 1) according to Eurocode 3 [1] are able to develop large plastic deformation before significant loss of strength under monotonic lateral loading [2]. However, recent research studies showed that this is not necessarily the case when they are subjected to cyclic loading [2], [3]. Araujo et al. [4] reported that the deformation capacity of steel members under cyclic loading is around 30% of the capacity under monotonic loading. The outcome of the European EqualJoints project [5] showed that full strength connections with large deep beams (e.g. IPE 600) achieve limited rotation where some of them did not reach the minimum requirement specified in AISC341-16 [6] for highly ductile connections. In the US context, the lower deformation capacity of steel wide flange

members subjected to standardized cyclic loading protocols has been the subject of several discussions in the literature [7]–[9]. The poor performance of some deep I-shaped sections commonly used in practice may raise a strong need for retrofitting of existing buildings. Thus, the development of new reinforcement methods to increase the rotation capacity of existing steel members of MRF is of paramount interest.

The use of bonded fiber-reinforced polymers (FRP) is well established for the seismic retrofitting of concrete and masonry structures. However, there is limited information about how they can be used to strengthen steel structures [10]. In the case of steel structures, there is a great research interest in using FRP for strengthening bridge girders and enhancing the fatigue performance of the structural components [11]–[13]. However, few studies focused on the effectiveness of the application of FRP to enhance the stability of members in the context of seismic design. El-Tawil et al. [1] tested steel double-channel built-up members subjected to flexural cyclic loading, where some of them were wrapped with CFRP in the plastic hinge region. The results demonstrated that the CFRP could increase the size of the plastic hinge and delay the occurrence of local buckling as well as lateral and torsional buckling. The drift at peak strength could be increased from 4.4% to 6.6%. Harries et al. [2] tested W-T steel specimens with stocky flanges and slender web retrofitted with GFRP subjected to monotonic and cyclic loading. The results revealed that the FRP controlled the web buckling, inhibited the formation of plastic kinking, and allowed the member to withstand a greater amount of monotonic and cyclic loading. Eglimez and Yormaz [3] tested beams with a high flange slenderness ratio (HEAA 400) under cyclic loading, where some of them were bonded with GFRP in the flanges. It was shown that the use of GFRP could moderately delay flange local buckling.

When premature fracture is prevented or delayed in MRF connections and the members are laterally restrained, local buckling controls the deformation capacity [3]. In beams, the local pattern starts by buckling in the free edge of the flange then followed by web buckling causing a significant drop of strength [3]. Several studies [3], [14], [15] showed that web slenderness plays an important role in the deformation capacity of steel members with deep section (i.e., $\frac{h}{b_f} \geq 2$, where h is the section height and

b_f is the flange width). Strengthening with bonded CFRP will share tension stresses thus reducing cumulative tensile plastic deformation and counteract plate bending deformation on the compressed plates caused by local buckling. Thus, it will delay the accumulation of plastic damage in every excursion imposed by the loading protocol which will potentially increase the rotation capacity. The effectiveness of this retrofitting technique will depend strongly on the adhesive toughness (i.e. strength and ductility) which will determine at which stage of the cyclic loading, CFRP debonding will occur.

This chapter investigates the effectiveness of the application of CFRP in enhancing the rotation capacity of deep steel beams by bonding CFRP on the web. A finite element model is developed in the commercial software ABAQUS. The model consists of a cantilever beam that represents the behavior of beams in MRF with length corresponding to the distance between the fixed end and the inflection point. Nonlinear geometrical instabilities, as well as the material nonlinear cyclic behavior of structural steel, are considered in the model. Debonding failure of the adhesive joint is simulated through the use of a cohesive zone model with traction separation law which simulates the elastic behavior, the damage initiation, and the evolution of damage through a softening behavior until reaching the complete separation of the adherend. Different adhesive types featuring different fracture properties are investigated. Finally, the cyclic behavior of the unreinforced control beam and CFRP retrofitted steel beams are compared.

10.2 Methodology

The purpose of this study is to investigate the cyclic behavior of steel I shaped flexural member reinforced with CFRP. Therefore, a high fidelity finite element model is developed in ABAQUS and subjected to symmetric cyclic lateral loading (i.e. SAC protocol) [16]. The model consists of a cantilever beam that is assumed to reproduce the behavior of the beam from one end to the point of inflection, which is assumed to be located at the mid-length of the beam. This reduced model approach has been broadly used in experimental and numerical studies [2], [4], [17]–[19].

Chapter 10: The application of CFRP to extend the deformation capacity of steel beams subjected to cyclic loading

The control unreinforced steel beam is an European IPE 400 profile with length $L = 1.35$, as presented in Figure 10-1 (a). One edge is fully restrained in the 6 degrees of freedom to ensure the fixed boundary conditions. The cyclic loading is applied on the other edge of the beam, which is restrained in the X-X direction to avoid out-of-plane displacements at the top of the specimen. Except for the CFRP patch, the reinforced beam, shown in Figure 10-1 (b), has the same geometrical and mechanical properties as the unreinforced one. CFRP patches are bonded on both sides of the web in the plastic hinge region. The dimensions of the CFRP patch are 400mm x 150mm x 3mm. This length is taken to ensure that the CFRP will cover the full plastic hinge region. Under flexural demand, one part of the web will be subjected to compression stresses and the other to tension. In order to avoid complex stress state on the CFRP patch, two separate plates are bonded on the web instead of one covering the full web face. Bonding the CFRP on the flanges may also contribute considerably to increase the stability of the member. However, this may increase the overstrength of the beam. In the case of beams part of steel MRF, it is of interest to increase the deformation capacity but not the strength which may compromise the weak beam-strong column condition. The configuration of using the composite patch on the web is intended to not provide more strength to the beam. The CFRP is expected to provide benefits in both tension and compression that may eventually contribute to an increase of the rotation capacity:

- (a) **Tension region**: The region subjected to large tension stress in the web experiences reduction of the thickness and increase of slenderness due to large plastic deformation, which reduces the buckling resistance when it will be subjected to compression in the next cyclic reversal. The CFRP patch will share the tensile stress with the web, which will delay the reduction of thickness and, therefore, will delay the development of web local buckling.
- (b) **Compression region**: The CFRP provides plate bending stiffness to the web which will act against the bending deformation induced by web local buckling.

The performance of the CFRP patch will depend strongly on the mechanical properties of the adhesives used for bonding. In this study, the influence of three different epoxy structural adhesives is investigated. The adhesives used are:

- (a) SIKADUR30: produced by SIKA® to be used for CFRP reinforcement in the construction industry. It is a rigid high strength structural adhesive commonly used in research related to the construction industry for concrete and metallic structures [20].
- (b) AF 163 2K: produced by 3M Scotch Weld. A high strength ductile adhesive used in the automotive industry [21].
- (c) Nagase Denatite XNR6852E-3: produced by Nagase ChemteX® (Osaka, Japan). A crash-resistant, tough epoxy adhesive, which is distinguished by its high strength, ductility, and resistance to impact. It can deform significantly before failure, allowing the bonded structure to deform plastically and to absorb the impact energy.

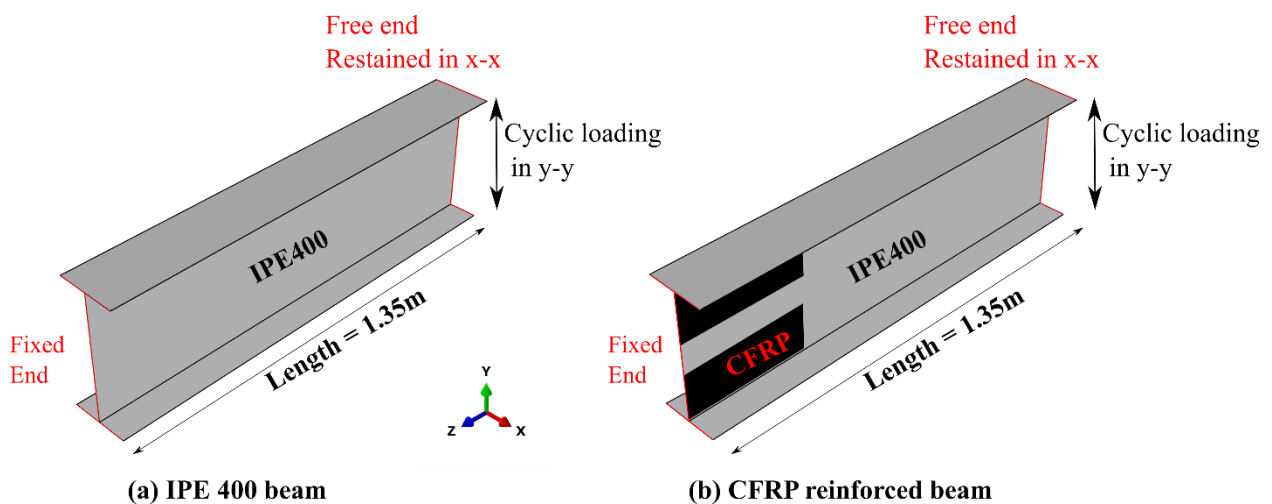


Figure 10-1: Beam model with and without CFRP reinforcement

10.2.1 Finite element modelling

10.2.1.1 Control beam without CFRP reinforcement

A Finite element model was developed in the commercial software ABAQUS which was used by the authors in a previous related study [3]. The beam plates are modelled using quadratic shell elements S4R with five Gauss integration points. After a mesh sensitivity study, the mesh size adopted is equal to $\frac{b_f}{12} = 15\text{mm}$, where b_f is the flange width. The length of the cantilever beam is taken as 1.35m,

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which corresponds to specimens of a future experimental campaign that will be conducted by the authors. Flange and web imperfections are included in the model considering the manufacturer's shape tolerances according to EN10034 [22]. The web imperfection is considered by adopting a bow shape having a maximum amplitude at the mid-depth to $\frac{h}{250}$, where h is the section height, and symmetric out of squareness flanges with a deviation for both flanges equal to $\frac{b_f}{250}$. More information can be found in [3].

The combined nonlinear isotropic/kinematic cyclic plasticity material model based on the Chaboche proposal [23] available in the ABAQUS materials library is adopted to capture the complex cyclic behavior of the material. The mathematical expression of the nonlinear isotropic component is presented in equation (1). It captures the cyclic hardening and/or softening behavior of the material through the enlargement or reduction of the Mises yield surface.

$$s^0 = s|_0 + Q_\infty(1 - e^{-be^{pl}}) \quad (1)$$

The nonlinear kinematic hardening component describes the translation of the yield surface in the stress space through the vector backstress α , to capture complex phenomena such as the Bauschinger effect. The backstress evolution law is the mathematical expression of the vector function that translates the centre of the Mises yield surface. Its mathematical expression is presented in equations (2).

$$\dot{\alpha}_k = C_k \frac{1}{s^0} (s - \alpha) \dot{e}^{pl} - g_k a_k \dot{e}^{pl} \quad (2)$$

The calibration of the plasticity model was conducted using experimental data on cyclic coupon tests provided by Chen et al. [24]. The parameters are presented in Table 10-1. More information can be found in [3], [25]. The finite element model is validated with experimental data from the literature. As shown in chapter 9.

Table 10-1: Nonlinear Isotropic and kinematic hardening parameters.

Material	σ_0 (N/mm ²)	Q_∞ (N/mm ²)	b	C (N/mm ²)	γ
S355	370	70	2	12640	91

10.2.1.2 Beam with CFRP reinforcement

Unidirectional pultruded S&P C- Laminate HM produced by the S&P reinforcement [27] are adopted in this study. The basic mechanical properties and the fiber content of the CFRP are shown in Table 10-2. The CFRP was modelled as an orthotropic elastic shell with the properties presented in Table 10-3. The engineering elastic constants were computed using the most widely adopted micro mechanical model [28].

Table 10-2: Properties of the CFRP laminates [27].

Property	S&P C-laminate HM
Density (g/cm^3)	1.6
Volume of fibres (%)	68
Tensile strength (MPa)	≥ 2800
Young modulus (GPa)	205
Elongation (‰)	13.5
Cross section (mm)	150 × 3

Table 10-3: Elastic properties of the CFRP laminates.

$E1$ (GPa)	$E2$ (GPa)	ν_{12}	G_{12} (GPa)	G_{13} (GPa)
205	9.179	0.35	3.53	3.53

The adhesive layer between the CFRP and the steel substrate was modelled using the cohesive zone modelling approach (CZM), which is the most widely used method to simulate the interfacial fracture in the adhesive layer [29]. In ABAQUS, there are two ways of using CZM, namely cohesive elements or cohesive surface, where the former approach consists of creating continuum finite elements with small thickness that will reproduce the behavior of the adhesive, and the latter method consists of creating a surface-to-surface contact between the CFRP and the steel with CZM properties. The use of the cohesive elements is impractical for this study due to the size of the beam. The consideration of finite elements with very small thickness compared to the beam size makes the numerical analysis computationally very expensive. Therefore, the cohesive surface contact modelling approach was adopted in this study. The cohesive behavior was modelled using the Traction Separation Law (TSL) which represents the relationship between stresses and relative the displacement between the CFRP and steel substrates. The TSL simulates the complete fracture process from the damage initiation and the evolution of damage through a softening behavior until reaching the complete separation. The widely used triangular shape TSL, shown in Figure 10-2, was adopted due its simplicity and conservatism [30]. Other shapes such as the trapezoidal shape, which is more suitable for ductile adhesives, can be found in the literature. However, this is beyond the scope of this study. Figure 10-2 schematically represents the different damage states that undergo the CZM model with its associated nomenclature where the symbols t and δ represent the stress and displacement, respectively. The subscripts n, s, t designate the normal, shear in the principal direction, and shear in the transversal direction, respectively. The parameter K is the penalty stiffness that will have no effect on the behavior if it is sufficiently larger than the transversal elastic modulus E_3 of the substrates. However, very large values may cause numerical problems [31]. Several researchers proposed a values for K . Zou et al. [32] suggested $4.5 \times 10^5 \leq K \leq 4.5 \times 10^8 \text{ N/mm}^3$, Camanho et al. [33] recommended to use 10^6 N/mm^3 , Turon

et al. [24] proposed $K = \frac{\alpha E_3}{t}$, where α is a coefficient that should be larger than 1. In this study, the penalty stiffness parameter K is taken as 10^6 N/mm^3 , which is the most common value used in the literature.

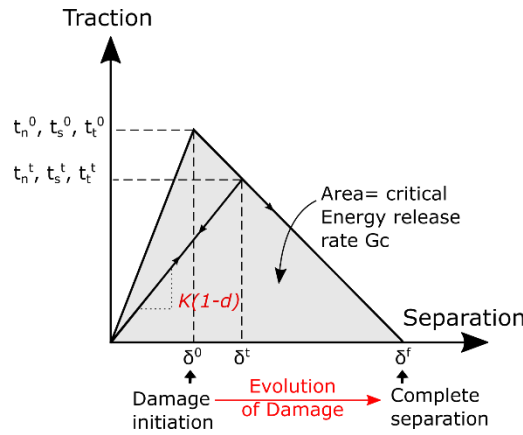


Figure 10-2: Traction separation law (TLS).

As stated previously, three adhesives are investigated in this study, namely one rigid adhesive (i.e. SIKADUR30) widely used in the construction industry and two tough ductile adhesives (i.e. AF 163 2K and Denatite XNR 6852 E-3) that are commonly used in the automotive industry. The properties of the adhesives are shown in Table 10-4, which were obtained by comprehensive experimental testing and numerical validations [20], [34]–[40]. The parameters G, σ_u, τ_u represent the shear modulus, ultimate tensile strength, and ultimate shear strength, respectively. The parameters G_{IC} and G_{IIC} denotes the critical strain energy release rates for fracture mode I and mode II, respectively. With particular care on the mesh size in the cohesive zone, which is defined as the distance from the crack tip to the point where the maximum cohesive traction is attained [29], the parameters presented in Table 10-4 can be used to calibrate the cohesive zone model and to accurately capture the interfacial behavior. The cohesive zone length “ l_{cz} ” for thin bonded joints is a material and a structural property that can be estimated using Equation (3) as proposed by [41].

$$L_{cz} = \left(E \frac{G_c}{(t^0)^2} \right)^{1/4} t^{3/4} \quad (3)$$

Where G_c and τ^0 are the critical strain energy release rate and the strength that can be for a single mode or mixed mode estimated by criterions available in the literature.

For an accurate simulation using CZM, it is important that the mesh size should be lower than L_{cz} . A mesh size between $\frac{1}{2}$ and $\frac{1}{5}$ of L_{cz} is commonly adopted [29], [33], [42]. Thus, the mesh size in the bonded region will be conditioned by the material properties of the adhesive presented in equation (5). For instance, for the Sikadur 30 under pure shear mode $L_{cz} = 1.97mm$, thus the mesh size needs to be less than $1mm$. For large models, such as the one used in this study, the mesh condition requires large computational resources that render the problem intractable. Turon et al. [29] noticed this issue and proposed a judicious engineering solution to overcome it by lowering the interfacial strength, as it does not have a strong influence on the results [43], which will result in a longer cohesive zone length and coarser mesh. The authors [29] proposed equation (4) to estimate the equivalent strength to input in CZM model given a mesh size l_e and number of elements in the cohesive zone N_e^0 . In this study, equation (4) was adopted to estimate the equivalent strengths $\underline{t}^0$ assuming 15mm mesh size and 3 elements in the cohesive zone.

$$\underline{t}^0 = \sqrt{\frac{9\pi E G_c}{32 N_e^0 l_e}} \quad (4)$$

The Quadratic nominal stress criterion (QUADS), shown in Equation (5) was used to simulate damage initiation.

$$\left\{ \frac{\langle t_n \rangle}{t_n^0} \right\}^2 + \left\{ \frac{t_s}{t_s^0} \right\}^2 + \left\{ \frac{t_t}{t_t^0} \right\}^2 = 1 \quad (5)$$

The symbol $\langle \rangle$ represents the Macaulay brackets that are used to show that a purely compressive stress state does not initiate damage. Once Equation (5) is satisfied, stiffness degradation will occur in the cohesive surface following the softening curve shown in Figure 10-3. The full failure of the cohesive

contact occurs when the critical fracture energy release rate, G_c , is reached. The mixed mode Benzeggagh-Kenane energy criterion [44] is adopted to estimate the critical energy release rate.

Table 10-4: Adhesives mechanical properties

Properties	SIKADUR30 [20]	AF 163-2K [35], [38]	Denatite XNR 6852E-3 [39], [40]
E (MPa)	9530	1520	1728
G (MPa)	3530	565	665
σ_u (MPa)	38	46.93	51.5
τ_u (MPa)	29	46.86	44.9
G_{IC} (N/mm)	0.7	4.05	9.2
G_{IIC} (N/mm)	1.33	9.77	51

1.1 Results and discussion

1.1.1 Overall behavior

Figure 10-4 shows the comparison between the buckling behavior of the bare steel beam and the CFRP reinforced beam (with AF 163-2K adhesive) at the second cycle of 40 mrad rotation imposed cyclic loading where both of them experienced local buckling and strength deterioration. The bare steel beam experienced flange and web local buckling combined with limited lateral and torsional buckling that can be observed from the non-uniform distribution of stress outside the plastic hinge region which clearly indicates out of plane bending. The CFRP reinforced beam experienced significant debonding of CFRP in the plastic region where at this stage it became ineffective since it is not bonded in the buckled region. The CFRP reinforced beam exhibits only flange and web local buckling with a more uniform stress distribution and longer plastic hinge. The CFRP prevented the lateral and torsional buckling at this level of demands. It is worth noting that this has been also observed by El-Tawil et al. [45] in physical tests conducted on double channel built-up members wrapped with CFRP in the plastic hinge region.

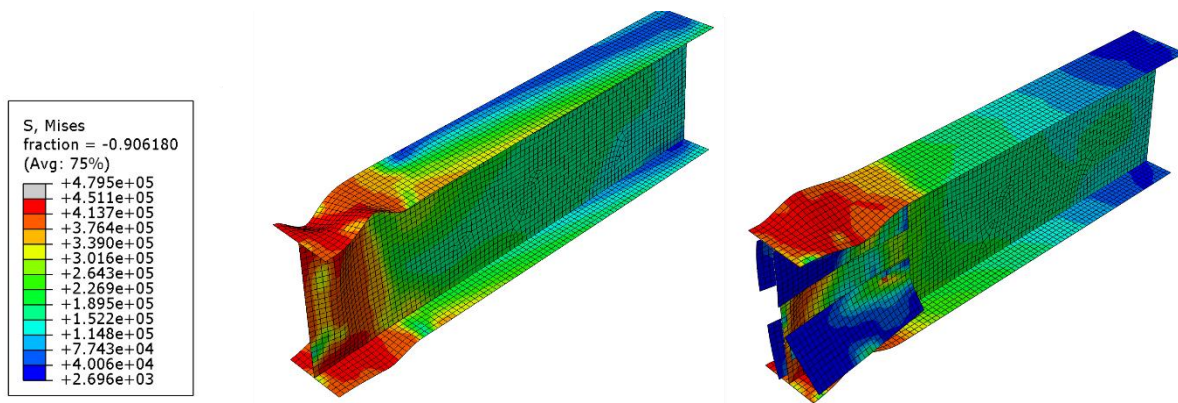
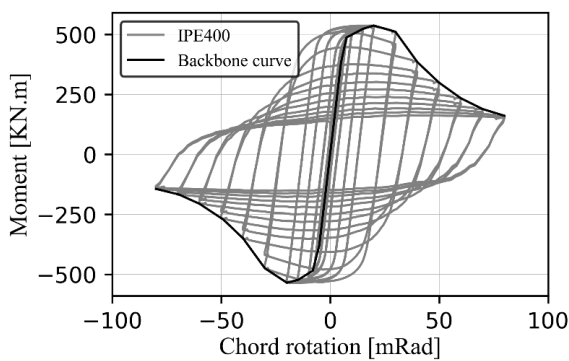


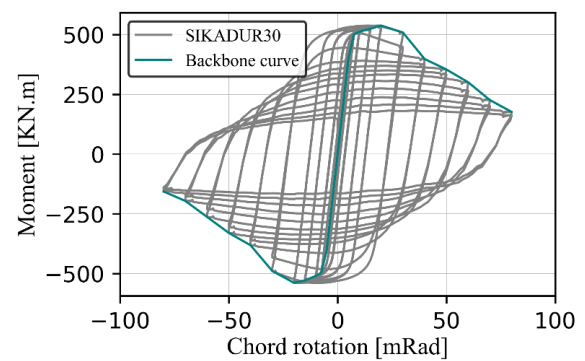
Figure 10-3: Buckling behavior of a bare steel beam and a CFRP reinforced beam (Unit kPa).

Figure 10-5 shows the moment-chord rotation curves and the first cycle backbone curve for bare steel beams and retrofitted ones. The chord rotation is equal to the imposed lateral displacement in the free end of the beam divided by the length. The moment-chord rotation hysteresis curves of the three reinforced steel beams are wider than the control steel beam. This clearly indicates that the CFRP contributes to higher energy dissipation, which is strongly desired in seismic design. Figure 10-5 (a-b) compare the positive and negative first cycle backbone curves of the control beam and the CFRP reinforced ones. As expected, the application of the CFRP did not increase the bending strength of the member which is very important as it would not compromise the weak beam-strong column condition in a steel MRF. The beam reinforced with rigid adhesive SIKADUR30 did not improve the cyclic performance considerably. It reached the same rotation at peak bending moment and failed at the same residual strength. However, the reinforced beam with SIKADUR30 benefited from a slightly lower strength deterioration rate as the post peak strength between $40\text{mrad} \leq \theta < 80\text{mRad}$ is higher than the control beam for both the negative and positive backbone curves. The beams reinforced using tough adhesives (i.e. AF 163 3K and Denatite XNR 6852E-3) exhibited an important enhancement of the behavior with an increase of 10mrad at peak moment, a lower strength deterioration and higher residual strength. Although this improvement may not seem significant, 10mrad of additional plastic rotation at peak moment can have a significant influence on the overall behavior of a steel moment-resisting framed structure. Besides, in the case of in a full-strength rigid MRF connections, by assuming that the components other than the beam have a negligible deformation, the enhancement obtained by

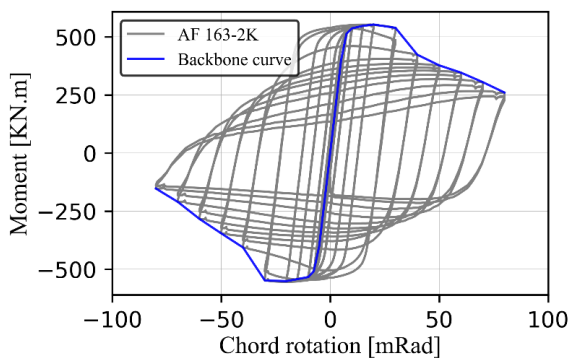
using CFRP can shift the classification of the connection from moderately ductile to highly ductile qualified connection to be used for seismic design. For instance, AISC 341-16 [6] requires a chord rotation of 40mRad at post peak strength of 80% yield moment (M_y). The unreinforced beam barely achieves this limit in the positive backbone curve shown in Figure 10-5 (a) and does not reach it in the negative backbone curve as shown in Figure 10-5 (b). Whereas the reinforced beams achieved both in the positive and negative backbone curves.



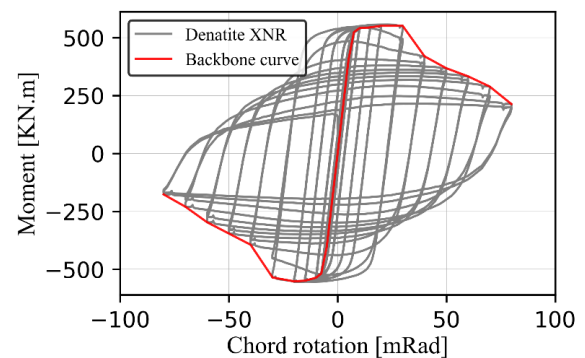
(a) Bare steel beam



(b) Steel beam patch with SIKADUR30

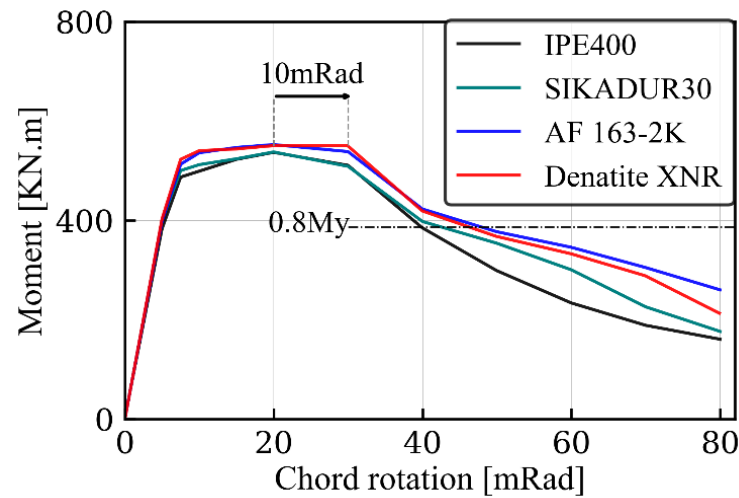


(c) Steel beam patch with AF 163-2K

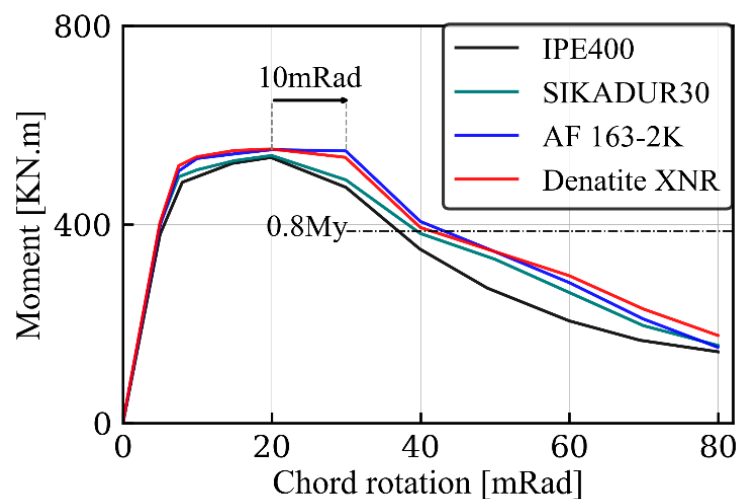


(d) Steel beam patch with Denatite XNR

Figure 10-4: Moment rotation cyclic curves and first cycle backbone curves.



(a) Positive first cycle backbone curves.



(a) Negative first cycle backbone curves.

Figure 10-5: Comparison between the bare steel beam and r reinforced ones.

1.1.1 Behavior of the CZM model

Figure 10-6 presents the comparison of the cohesive behavior of the reinforced beams bonded with the three different adhesives considered in this study. For this purpose, the CSDMG parameter, which is a ratio that represents the damage state of the adhesive layer, is adopted, where 0 indicates an undamaged adhesive layer and 1 a fully debonded adhesive layer. In the three cases, the damage started at very

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early stages (5mrad) at the free ends of the CFRP. This is a very common issue in adhesively bonded joints where high peeling stress develops at the free ends. In order to reduce this effect, several solutions can be found in the literature such as tapering the CFRP ends, using adhesive fillets [46]–[48].

The CFRP patched with the rigid adhesive SIKADUR30 reached more than 90% of damage at very early stages of the loading (i.e. 10mrad). This is the main reason that justifies the limited improvement of the cyclic performance observed in the previous section. In the case of the AF 163 2K adhesive, the damage initiated from the free ends and propagated towards the buckled region. Significant debonding started at 30mrad and complete debonding in the buckled region occurred at 40mrad . Similar behavior was observed with the other tough adhesive DENATITE XNR but with lower damage level since it is much tougher than the AF 163 2K.

As shown in the previous section, the difference between DENATITE XNR and AF 163 2K performance is not significant. This might be influenced by two reasons:

- (a) The imposed deformation demands by the loading are very high which cause debonding before buckling begins. Therefore, the CFRP is ineffective at this critical stage and a significant improvement on the behavior cannot be achieved.
- (b) The triangular shape of TSL used in this study maybe very conservative especially for the case of DENATITE XNR, which is a very ductile adhesive.

With the reinforcement configuration used in this study, significant deformation demands experienced the CFRP and the adhesive which led to the damage of the adhesive layer to initiate at very early stages. Different configuration by patching the CFRP on the flanges will reduce deformation demands caused by local buckling. Considering that the CFRP in the flanges will debond anyway at around 30mRad or before. The CFRP patch on the flanges may not increase the strength of the member and hence it is recommended that this configuration is investigated in future studies.

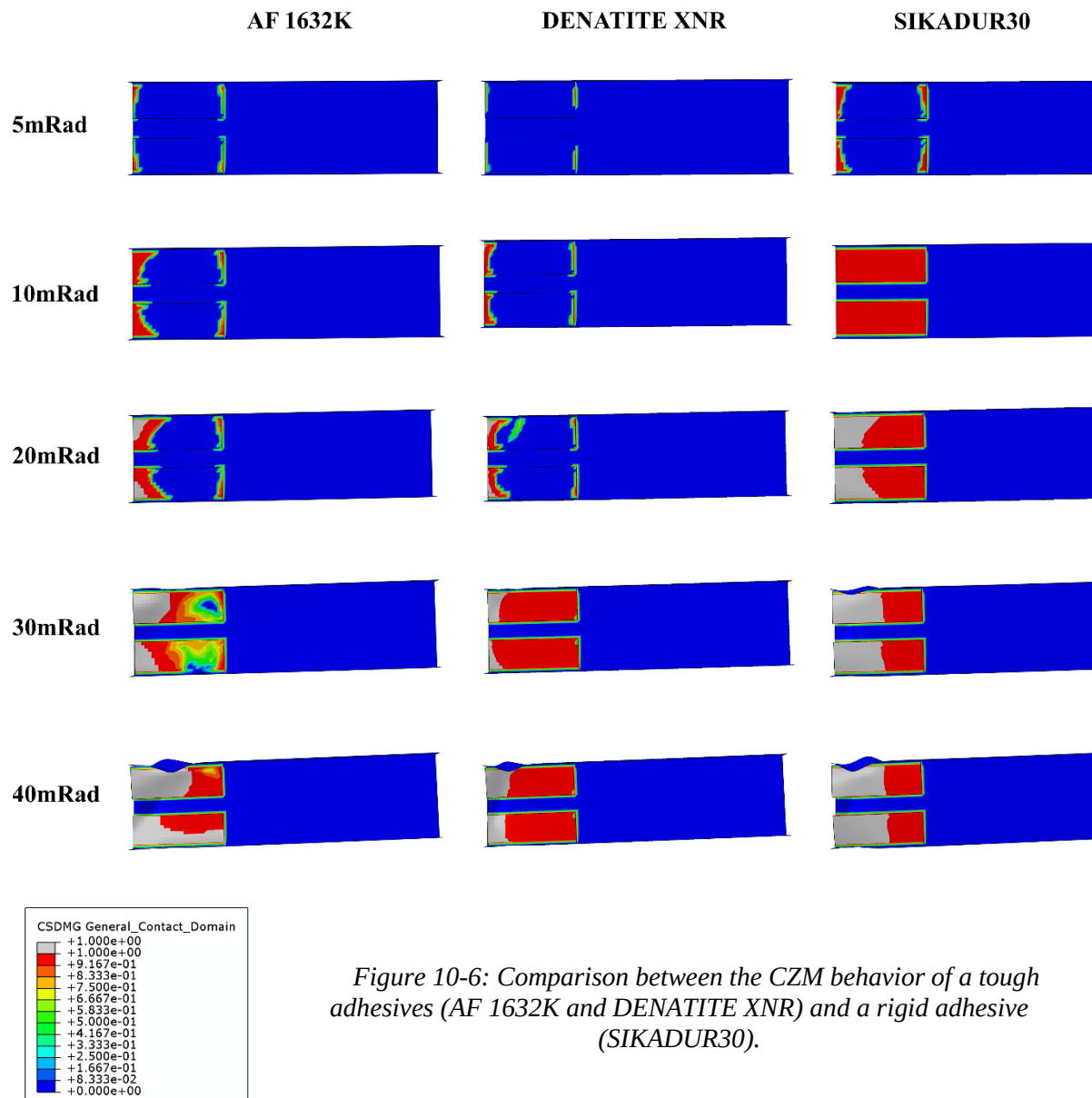


Figure 10-6: Comparison between the CZM behavior of a tough adhesives (AF 1632K and DENATITE XNR) and a rigid adhesive (SIKADUR30).

1.2 Conclusions

This chapter investigated the cyclic performance of I shaped beams patched in the web with CFRP using high fidelity finite element analysis. The modelling methodology of the CFRP patch using cohesive zone modelling approach is thoroughly detailed which can be useful for future research related to civil engineering structures since these methods are typically used in research in the automotive and aerospace industries.

The main conclusion and recommendations can be summarized as follows:

- (a) The beneficial effect of the CFRP patch was clearly evident in several aspects such as the enlarged hysteresis curves, elongated plastic hinge region, and the increased the rotation capacity.
- (b) It is clear that the use of tough adhesive is inevitable for seismic design applications where large plastic deformations are involved. The adoption of rigid adhesives commonly used in construction practice does not provide any significant benefits in what concerns increase in the deformation capacity of steel members.
- (c) Debonding started in the free edges early stages, thus techniques for reducing the peeling stress such as tapering the CFRP, or applying some compressive stress with clamps should be considered in future research studies.
- (d) The difference between the two tough adhesives could not be clearly observed. This is mainly due to the high plastic deformation experienced by the web. However, the use triangular TLS is conservative for tough adhesives used in this study, other more adapted models such as the trapezoidal will provide a higher performance of the adhesive.

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CHAPTER 11: Conclusion and future works

11 Conclusion and future works

11.1 Overview

The preservation of old metallic infrastructures, such as bridges, train stations, high-rise steel buildings, etc., is an important issue for the sustainability of modern cities. A large number of these structures have been built more than 50 years ago using conventional design codes (if any existed) that do not meet the current requirements of current codes. Aging problems such as corrosion and fatigue have been particularly encountered in several bridges due to their exposure to harsh environments and being subjected to repeated loadings that are increasing in intensity and frequency over time. The resilience of these old infrastructures after natural hazards such as earthquakes is an open issue. The replacement of these existing structures is not economically, environmentally, or socially possible. Therefore, the development of strengthening methods to upgrade the structural performance of old metallic structures is of paramount interest.

The use of bonded carbon fiber polymers (CFRP) for strengthening old metallic structures is very attractive due to the easiness of its application and the potential effectiveness of this technique. In the last decade, a lot of research efforts have been devoted to this issue. However, the application of this technology in practice is still at its early stages. Whereas, for concrete structures or other industries such as marine and aeronautic, the application of bonded CFRP to repair deficient structural elements is routinely practiced. To implement this technology in practice for old metallic, experimental evidence that shows the effectiveness of this technique needs to be demonstrated and simplified design procedures need to be developed and implemented in design codes.

This thesis covers a wide range of subjects related to the application of CFRP to strengthening metallic structures. It was mainly divided into three parts, where part I focuses on the local behavior of CFRP/steel adhesive joints; Part II: on fatigue strengthening of metallic structural details using CFRP; and part III: on the application of CFRP to extend the deformation capacity of steel members subjected to cyclic loading for the seismic design of steel structures.

11.2 Main contributions

The main contributions of this thesis can be summarized as follows:

Part I: On the characterization of CFRP/Steel adhesive joints

Chapter 2: a state-of-the-art review on the behavior of CFRP/Steel adhesively bonded joint was conducted where:

- Most of the research studies used brittle adhesives that may not provide the best performance for steel structures. Instead, a more ductile adhesive should be used.
- A critical discussion on the type of specimens used in the literature is provided. Where the need to use more representatives is highlighted.
- The empirical models based on the assumption of only mode II is not realistic.
- The influence of fatigue loading on the behavior on the adhesive joint behavior is discussed.

Chapter 3: An experimental and numerical study on the behavior of CFRP/Steel adhesive joint was presented, where a large parametric study that includes several variants such as the adhesive type, adhesive thickness, CFRP length, CFRP type, and the steel thickness was conducted. It was shown that:

- Brittle adhesives commonly used in the construction industry have achieved very low strength compared to the ductile adhesives available in the market.
- The influence of the steel thickness on the behavior of the adhesive joint is significant. The stress concentration occurs in the middle of the joint for components with thick steel plates and occurs at the end of the overlap for thin steel plates. This is mainly due to the plastic deformation that occurs in thin steel plates. This issue is very important and highlights the need to use more representative specimens in real situations.
- Recently developed, acrylic-based ductile adhesives that do not require particular surface treatment achieved very high strength when using very simple surface treatment (sandpaper).

- Ductile adhesives that combine high strength and ductility have very high fracture toughness. Therefore, the traditionally desired cohesive failure that occurs in the adhesive layers will become unlikely when using this type of adhesives. Instead, CFRP delamination becomes more dominant. Numerical and analytical models that consider this failure mode needs to be developed.
- A simplified design method based on joint efficiency is proposed for design purposes to avoid the cumbersome analysis of the adhesive joint for engineers of practice.

Chapter 4: Cost-effective CFRP/Steel adhesive joint for the construction industry

- It was shown in chapter 3, it was shown that the ductile adhesives are more appropriate for the CFRP/steel adhesive joints. However, these adhesives were initially developed for the automotive or the aeronautic industries and are very expensive for the construction industry where the size of the structural members can be significant. Therefore, a cost-effective CFRP/Steel adhesive joint is proposed by using a limited amount of the ductile adhesive in the stress concentration zone and the brittle (with lower cost) in most of the area of the joint. The bi-adhesive joints achieved very high strength which is comparable to the performance of the joints with ductile adhesive alone.

Part II: The application of CFRP to extend the fatigue life of metallic structural details.

Chapter 5: A literature review on the fatigue strengthening of old metallic bridges using CFRP is presented, where:

- The state of existing old metallic bridges is discussed.
- Case studies of bridges strengthened with CFRP are presented.
- An extensive literature review of studies related to the application of CFRP to extend the fatigue life of metallic components is conducted. It was shown that most studies in the literature focus on the repair of cracked metallic components and little is known about fatigue strengthening of non-cracked metallic structural details, where the fatigue degradation of the CFRP might cause an issue due to the relatively long fatigue life.

Chapter 6: Statistical fatigue assessment of CFRP retrofitted steel structural details is conducted, where:

- Experimental fatigue data of non-cracked structural steel details reinforced with CFRP are collected from the literature and analyzed using statistical methods.
- It was shown that the CFRP strengthening was very effective at lower stress levels ($< 200\text{MPa}$) where the fatigue data tend to converge towards a fatigue limit. Whereas, at high-stress levels, CFRP strengthening provided no extension of fatigue life. This was attributed to the early debonding of the CFRP patch due to the fatigue loading.
- Probabilistic S-N curves based on three parameters Weibull distribution is proposed. This model is recommended for CFRP retrofitted structural details due to its flexibility to fit data.

Chapter 7: An approach for predicting the fatigue life of CFRP retrofitted metallic structural details

- In order to consider the fatigue degradation of the CFRP patch while calculating the fatigue life of metallic structural elements reinforced with CFRP. A simplified approach is proposed based on a fatigue phenomenological residual stiffness model of the composite patch and fatigue damage accumulation rules of the metallic structural detail. The approach is fairly simple compared to existing numerical methods and has been validated with an experimental fatigue dataset collected from the literature on aluminium plates featuring a hole in the middle and patched on both sides with CFRP sheets.

Chapter 8: Fatigue crack growth modelling for cracked small scale structural details repaired with CFRP

- Since a lot of experimental research was published in the literature on the application of CFRP to repair cracked structural details repaired with CFRP. A database of all experimental data available in the literature was developed.
- An analytical fatigue crack growth model is proposed to model the fatigue crack growth of CFRP repaired structural details. The model is fairly validated with experimental data.

Part III: The application of CFRP to extend the deformation capacity of steel members subjected to cyclic loading for the seismic design of steel structures.

Chapter 9: Deformation capacity of I-shaped steel beams subjected to cyclic flexural loading for the seismic design of steel structures

Recent research showed that I-shaped steel beams, which were traditionally believed to be highly ductile and capable to dissipate a lot of energy through the formation of plastic hinges during strong earthquakes, do not achieve the expected deformation capacity when subjected to cyclic loading. Therefore, firstly, it was important to characterize the behavior of these structural members in this chapter and then propose a retrofit solution for the members that had a poor performance in the next chapter. Therefore:

- Characterization of the behavior of I-shaped steel members with European profiles subjected to cyclic loading was provided.
- The empirical formulation for the estimation of the rotation capacity is provided for the update of the Eurocode 8 part 3.
- It was shown that Deep beams having a high width/height ratio achieve a low deformation capacity due to the web buckling.

Chapter 10: upgrade of the deformation capacity of I-shaped steel beams using CFRP

- A retrofit solution is proposed to increase the deformation capacity of steel I-shaped beams subjected to cyclic loading by patching the CFRP in the web where local web buckling was observed in the previous chapter.
- The results revealed that the CFRP patch could increase the deformation capacity of the steel beams considerably.
- A comparison between different adhesive types was conducted. It was found that it is important to use very ductile adhesives.

11.3 Future works

CFRP/Steel adhesive joint:

As was shown in chapters 3 and 4, recently developed ductile adhesives that combine strength and ductility provide very high performance. As most previous research studies used brittle adhesives. More experimental studies using ductile adhesives are recommended. As stated previously, due to the high toughness of the ductile adhesives, cohesive failure is unlikely to occur and CFRP delamination becomes dominant. Therefore, numerical and analytical models that consider delamination failure mode are highly recommended for future research. Also, reinforcement techniques to increase the toughness of the CFRP laminates may increase the strength of the CFRP joint. Research towards the development of joint efficiency catalogs that can classify the joint strength based on the adhesive type, CFRP length, and surface preparation is highly recommended.

The application of CFRP To extend the fatigue life of metallic components:

Research towards the improvement of the approach proposed in chapter 7 that considers the fatigue degradation of the CFRP patch is highly recommended. This can be done by developing a more advanced fatigue stiffness degradation model based on experimental data.

Due to the lack of experimental data, the effect of the adhesive type on the fatigue life of the composite patch is not clear. Future research on this issue is highly recommended.

More experimental works on the fatigue behavior of old metals repaired with CFRP patch are highly recommended.

The application of CFRP strengthening of steel structures for seismic design context:

The application of CFRP has great potential in the seismic retrofit of steel structures. Properly designing CFRP patch using adequate adhesives (highly ductile) can bring many benefits to a structural member, such as additional confinement in case of hollow sections, promoting more uniform stress distribution, and

delaying local buckling by providing additional stiffness. This can result in higher ductility of the member which is highly desirable for seismic design applications. Therefore, studies on CFRP strengthening for seismic applications are highly recommended.