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Fatigue Behaviour of Steel Bridge Joints Strenghtened with FRP Laminates

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Abstract

One of the most relevant concerns about steel and metal bridges stands on the repair and rehabilitation of existing structures. In fact, the remaining service life of steel bridges is limited by fatigue damage and in order to ensure the safety of these bridges it is often necessary to inspect the structure in order to discover the presence of fatigue cracks. New reinforcement techniques are needed, in order to prevent fatigue cracking and to increase bridge safety at the same time. According to the contemporary knowledge on the matter, two alternatives of rehabilitation are given: the traditional interventions and the use of carbon fibre composites. In this paper, in order to explain the appropriate use of these techniques, the most common knowledge on the matter are presented and explained, focusing in particular on the real scale testing results coming from the last decades research applications related to steel bridge engineering. This study should in this way be useful for future research applications and for bridge engineering real case rehabilitations.

Keywords: fatigue, steel, bridge, rehabilitation, FRP

1. Introduction

The remaining service life of steel bridges is limited by fatigue damage and in order to ensure the safety of these bridges it is often necessary to inspect the structure in order to preclude the presence of fatigue cracks. Damage due to cyclic loadings deals with a fatigue failure that proceeds by gradual enlargement of fatigue cracks during the tensile part of the loading cycle; once a crack reaches a certain critical size, catastrophic failure ensues. The process of fatigue crack growth is evidenced by the formation of characteristic striation patterns on the fracture surface called beach markings, which are followed by the more irregular pattern characteristic of fast fracture. In order to maintain these bridges in service new reinforcement techniques are needed, in order to prevent fatigue cracking and increase bridge safety at the same time. The use of carbon fiber composites (CFRP) for structural reinforcements has shown increasing promise. Aside from the very high tensile resistance, composite materials have also demonstrated a very high fatigue resistance, which makes them particularly appropriate for the reinforcement of steel structures subjected to fatigue loads. Degradation of steel structures, such as old industrial buildings and bridges, and increased load requirements have led to the need for structural rehabilitation and strengthening. Another rehabilitation procedure stands on traditional applications as HS-bolting or welded steel plates to the original structure, sometimes used as a strengthening technique: some negative aspects such as the increase in permanent loads, difficulty of applications, and problems due to corrosion and fatigue have been demonstrated to affect also the new members or connections. On the contrary, fiber reinforced polymer (FRP) materials have a high strength-weight ratio, do not give rise to problems due to corrosion, are extremely manageable and are commonly used for strengthening structural existing members (Lua et al., 2005; Yuana et al., 2004; Jao et al., 2005; Pellegrino et al., 2002), but they have only been infrequently studied and used for strengthening steel elements and, above all, steel-concrete composite elements. Several studies on the FRP strengthening of concrete structures have resulted in the first design guidelines for concrete structures strengthened with externally applied FRP. American ACI 440-02 (ACI, 2002), European fib bulletin 14 (FIRB, 2001) and Italian Recommendations (CNR, 2004) are examples of such guidelines. Although externally bonded FRP strengthening for metal structures is a rapidly developing technique (Cadei et al., 2004), guidelines for the reinforcement of steel structures with FRP do not currently have a comparable degree of reliability. Some examples of guidelines for the design and construction of externally bonded FRP systems for strengthening existing metal structures are the ICE design and practice guide (Moj, 2001), CIRIA Design Guide (CIRIA Design Guide), US Design Guide (Schnerch et al., 2007) and document CNR-DT 202/2005 (CNR, 2005), which can currently be considered as a preliminary study. The benefits of composite strengthening are shown in some significant case studies like the successful strengthening of a steel bridge on the London Underground (Moj, 2007). The benefits of strengthening large cast-iron struts with Carbon FRP composites in the London Underground are illustrated in (Moj, 2006). Significant work in this area of research was also developed by (Tavakkolizadeh et al., 2003a; Tavakkolizadeh et al., 2003b; Shaat et al., 2006; Shaat & Fam, 2004; Sen et al., 1995; Miller et al., 2001). A state-of-the-art review on FRP strengthened steel structures was recently developed by Zhao and Zheng (2007). Main approaches proposed in the literature to study the phenomenon of delamination (Buyukozturk et al., 2004) are based on linear elastic analysis of internal stresses (Taljsten, 1997) and linear elastic fracture mechanics (Lenwari, 2005; Lenwari et al., 2006; Colombi, 2006). Other significant approaches to the determination of stresses in the adhesive layer are described in the CIRIA Design Guide (Cadei et al., 2004) and in the work of Deng et al. (2004). The fundamental work about stresses at the interface was reported in Smith and Teng (2001) and in Stratford and Cadei (2006). These theories have not been validated with a sufficient number of experimental results, due to the few experimental data available in the literature. One of the first studies on the flexural behaviour of steel beams reinforced with carbon-fibre reinforced polymer composites is that of Moy and Nikoukar (2002). Very recent works about strengthening steel structures using FRP are those of Schnerch and Rizkalla (2008) and Rizkalla et al. (2008). Current design guide lines propose a method to predict the flexural behaviour of FRP strengthened elements based on classic equilibrium and strain compatibility, usually within the elastic range of the materials. FRP laminate is always considered only under the tensile flange current in-depth information about the non-linear behaviour of FRP strengthened steel-concrete composite elements is scanty. Concerning the phenomenon of fatigue in existing steel bridge member, some works could be noticed in literature, dealing in particular with the assessment of riveted connections, both by full scale tests, or by step level procedure, or finally applied on real case applications (Pipinato et al., 2012a; Pipinato et al., 2012b; Pipinato et al., 2012c; Pipinato et al., 2012d; Pipinato et al., 2011a; Pellegrino et al., 2011; Pipinato, 2011; Pipinato et al., 2011b; Pipinato et al., 2010; Pipinato & Modena, 2010; Pipinato, 2010; Ppinato et al., 2009; Pipinato et al., 2008). This paper deals with the analysis of experimental tests developed in a variety of steel joints repaired with FRP laminates. The purpose of the study is to investigate three main groups of test: (a) tension tests on single plates with FRP reinforcement; (b) tension test on double plates with FRP reinforcement; (c) bending tests on reinforced full scale beams with FRP reinforcements on the lower flange. SN curves are provided in order to check the correspondence of experimental results and coded provisions.

2. Existing Metal Members

Iron bridges have been built since the industrialisation time period, which was the end of the 19th century, so old metal bridges could imply to assess a structure with an average age of more than 100 years. Several bridges even exist since the mid of the nineteenth century, leading to an age of more than 150 years. The early metal bridges in the 19th century were fabricated with cast iron or puddle iron (wrought iron). The manufacturing of puddle iron started in the beginning of the 19th century. Since it had a lower carbon content than cast iron, going along with a better ductility, it allowed forging and an easier workmanship. Yet towards the end of the 19th century puddle iron was superseded by mild steel that obtained higher qualities concerning the chemical analysis and cleanness of the steels as well as better technological material properties (e.g. weldability, strength). The steel production and construction technology (bolting and welding of joints) developed quickly but testing methods to examine relevant properties as toughness, fatigue, corrosion etc., were missing completely. Such testing methods were developed much later in the 20th century and related to modern steels rather than old steels of existing structures. Therefore only fragmentary knowledge exists concerning iron materials of the early times, complicating the handling and assessment of old metal structures. Already at the beginning of the 20th century metal bridges were built mainly with mild steel. It should be also considered that before 1910, there was little standardization in the industry: each steel producer used his own recipe and rules. This resulted in a wide variety of metals used, regarding the chemical and mechanical properties. Railway authorities, for instance, adopted specific rules only from 1900 in order to specify the materials to be used for iron railway bridges. In order to assess existing bridges it is essential to know the material properties and their characteristics. Constitutive materials could be: grey cast iron, wrought iron, mild steel and the contemporary steel product. Grey cast iron takes its name from the colour of the fracture face, has a carbon content of 1.5-4.3% and 0.3-5% silicon plus manganese, sulphur and phosphorus. It is brittle with low tensile strength, but is easy to cast. The characteristics of the material are good in compression but poor in tension. Thus, the structural parts where often designed to be in compression as arches, columns etc. The graphite flakes caused a significant brittleness of the material. Internal cracks could easily occur and propagate along the flakes when the iron was subjected to tensile stresses, giving poor ductility properties. This material has a good wear resistance and damping abilities, absorbing vibrations and noise, even if it is brittle and has demonstrated a poor resistance to impact and shock. Cast iron is not suitable for welding due to its high carbon contents, which can lead to brittle cracks in and around welded joints. Wrought iron (or puddle steel) was the first structural steel until it was replaced by (mild) steel, in the end of the 19th century. It has a low carbon content, an high amounts of phosphor and nitrogen making the material brittle and escalating the ageing process. The microstructure is non homogenous due to the manufacturing process producing inclusions of sulphides and oxides. This led to anisotropy of the material which is especially bad in the thickness direction due to the arrangement of the inclusions and the influence of the rolling. As a matter of fact, the mass production of steel started with the Bessemer process 1856, followed by the Martin-Siemens process 1867 and the Thomas-Gilchrist process 1878. Most of the old metal bridges still used today consisted of steel produced with one of these processes. Production of steel in the end of the 19th and the beginning of the 20th century were conducted with a technique called chill module casting. The chill module casting was performed by pouring the steel from the oven in to a chill module to cool down before rolling of the steel. The cooling process in the module started from the borders, with high temperatures in the middle. During the cooling process almost pure steel formed at the borders and unwanted alloys and impurities increases towards the centre of the melt (depending on thermal properties). This manufacturing process lead to the formation of impurities and blisters increased in the middle of the steel, and for this principle reason steel produced during these circumstances are not considered appropriate as construction steel today. Moreover, an high concentrations of unwanted compounds formed in the middle of the steel that drastically lower the quality, so laminates manufactured with this technique are characterized by steel with very good qualities at the surface while the centre of the plate will be steel with brittle properties. At the same time, they are not suitable for welding, due to a differentiated toughness, and for this reason cracks can originate due to the residual stresses from the heat affected zone of the weld. Finally, contemporary steel materials, are coded and produced according to general standards and methodologies, for e.g. in Europe according to EN 1993-1-1 (2005), characterized by different material properties as the tensile resistance, the fracture toughness, the ductility requirements.

3. FRP Materials

The principal fibres that are used in FRP plate bonding to metallic structures are carbon fibres, aramid fibres and E-glass fibres: a summary of the indicative fibre properties is given in Table 1. Carbon fibre is characterized by excellent fatigue resistance, and do not suffer from stress rupture compared with glass or aramid fibres. Carbon fibre properties depend on the structure of the carbon used, so typically they come defined as standard, intermediate and high modulus fibres, depending from their structural behaviour. Aramid fibres have the highest strength to weight ratio (for e.g. Kevlar) and is characterized by similar tensile strength to glass fibre, but can have modulus greater at least two times; it could allow significant energy absorption but, compared to carbon, it is lower in compressive strength and has poorer adhesion to the matrix, finally it is also susceptible to moisture absorption. E-glass accounts for the larger part of the glass fibre use and is used mainly in a polyester matrix: the ‘E’ in E-glass stands for ‘electrical’ and was intended to indicate that the material has low electrical conductivity. It is commonly the the cheapest glass fibre and is therefore the preferred material in the construction market. Two types are available one contains boron, the other boron free: the corrosion resistance of E-glass without boron is approximately seven times the corrosion resistance of the boron-containing E-glasses.

4. Experimental Tests

The first group of experimental tests investigated, is represented by FRP reinforced steel plates under tension forces (Figure 1) (Jones & Civjan, 2003). Examined details were notched (N) or hole (H) provided: N11 not reinforced; N21 is made of 4 Sika Wrapex 103C resin on one side; N31 is made of Sika Wrapex 103C resin on two sides, L=255 mm; N41 is made as N31 but with a smoothed resin layer; N35 is made as N31 but with an unbalanced resin mix; N51 is made as N31 but with L=380 mm; N22 is made as N21 but with MBRACE CF130; N36 is made as N31 is made as N21 but with MBRACE CF130; H11 is not reinforced; H21-22 is made of Sika Wraplex 103 C on two sides, L=255 mm; H41-42 as H21 but with Sika Carbodur. In this test the FRP application caused eccentricity causing bars curvature, and as a result the performance doesn’t grow up if compared to the unreinforced test. Three types of CFRP have been used with diverse elasticity module, and varying thickness: from these tests it could be obtained that the best performance is obtained by using the CFRP with a lower E module (e.g. Sika WrapHex 103C). Moreover, from the obtained comparison on SN curves, an extended life has been obtained for: holed specimens, accurate preparation of the base layer, longer CFRP laminates. Category details, even if approximate, stands in the range C=140-160 for notched specimens, and on C=160 for holed specimens. A similar experimental testing has been provided as reported in Figure 2 (Monfared et al., 2008): in this test, similar specimens of the aforementioned test has been used. Variable parameters have been used, as: a reinforcement on one or two sides, and the result was that a single side specimen has a longer fatigue life; an accurate sanded surface preparation, belonging to the result that the surface preparation doesn't provide significant improvements. While in the reinforced specimens the failure stands on the reinforment, in the not-sanded specimens the CFRP delaminates. In Figure 2 it could be noticed a tendency line for the failure specimens, belonging to the following detail category: C=78 for not-reinforced specimens, and C=82 for reinforced specimens. Another similar application has been developed (Zheng & Lu, 2006) and reported in Figure 3: in this study the reinforced detail on both sides highlighted a longer fatigue life, which is improved also by higher stiffness value, depending on the E module and on the thickness. The detail failed for delamination. The following detail categories were obtained: C=63 for not-reinforced specimens, and C=102 for reinforced specimens. The second group of experimental investigations analyzed, is represented by double strap FRP reinforced steel plates under tension forces (Figure 4) in this case the failure mechanism is represented by delamination for SDS specimens, and CFRP failure for the DDS, beyond the C=40 (Matta & Karbhari, 2005); in the second case (Liu et al., 2005), the failure mechanism is influenced by the CFRP module; with a normal module CFRP delamination prevail, while with higher module CFRP the composite laminates fails. Other studies, presents some analysis on existing bridge steel members specimens of the first of '900, taken out of service and repaired with CFRP: in the first cases (Bassetti, 2001) the CFRP stiffness is varying from SikaCarboDur S512 with a lower module (E=155 GPa) and laminates of 1.2 mm thick, to higher module (E=210 GPa) and laminates of 1.4 mm thick; the mechanical strength resulted to be not influenced from the quality of the resin, while the pre-tension of the CFRP could grow-up to 20 times the fatigue life; failure resulted to remain in the delamination of the CFRP. Other observations has been done in (Täljsten et al., 2001) in relation to the propagation speed growth of the CFRP, which has been demonstrated to depend from the pretension force, from the stress reversals and pre-tension, and from the importance of the cleaningness of the surface; as a result, pre-tensioned specimens could stop the crack propagation into the very high cycle phase: for e.g. a pre-tension specimen has been tested up to 16x106 cycles, belonging to an extended life raising eight times the category detail life. Moreover, the crack propagation has been observed to be symmetric except if a delamination raise up. In the graph reported in Figure 6 two groups has been presented: in red not-reinforced details, in blue reinforced details, with the further specifications denoted in the legenda. From these comparisons, it could be noticed some SN tendencies, such as C=100 for virgin specimens, C=106 for not reinforced beams, and C=152 for reinforced beams. The third group of experimental investigations analyzed, is represented by bending tests on steel beam (Figure 7): four riveted beams have been tested by bending fatigue tests. Reinforcing technique are for e.g. represented by cover-plating or rivet/bolt substitution or by stop-holes: the limits observed in the testing results are that these are applicable for small cracks, and could be applied in the first propagation phase; moreover, these technique are limited to local repairing. CFRP interventions are otherwise reported in the following Figure 8, in which not reinforced and a corresponding CFRP reinforced section has been tested; in particular for the Q4 specimen, it should be observed that the test has been stopped at 20 milion of cycles without failures nor crack propagation, while in the Q5 and Q8 specimens, even if large crack openings has been found before testing, these has been limited in the growing phase, and no CFRP reinforcement has been found at the end of the test. To sum up all the tests analyzed, the principles data have been reported in Tables 2 and 3.

Table 1. Typical fiber properties used in structural interventions

Property	Ultra-high-modulus carbon fiber	High strength carbon fiber	High-modulus carbon fiber	Aramid fiber Kevlar 49	E-glass fibre
Specific weight	2.12	1.80	1.80	45	2.56
Modulus of elasticity (GPa)	620-935	230	390	130	70
Tensile strength (MPa)	3600-3700	3400	2900	3000	-
Strain to failure (%)	0.6	1.48	0.74	2.3	-
Coefficient of thermal expansion (10^{-6} °C)	-	-1.0 to +0.4	-1.0 to +0.4	-5.2	4.9

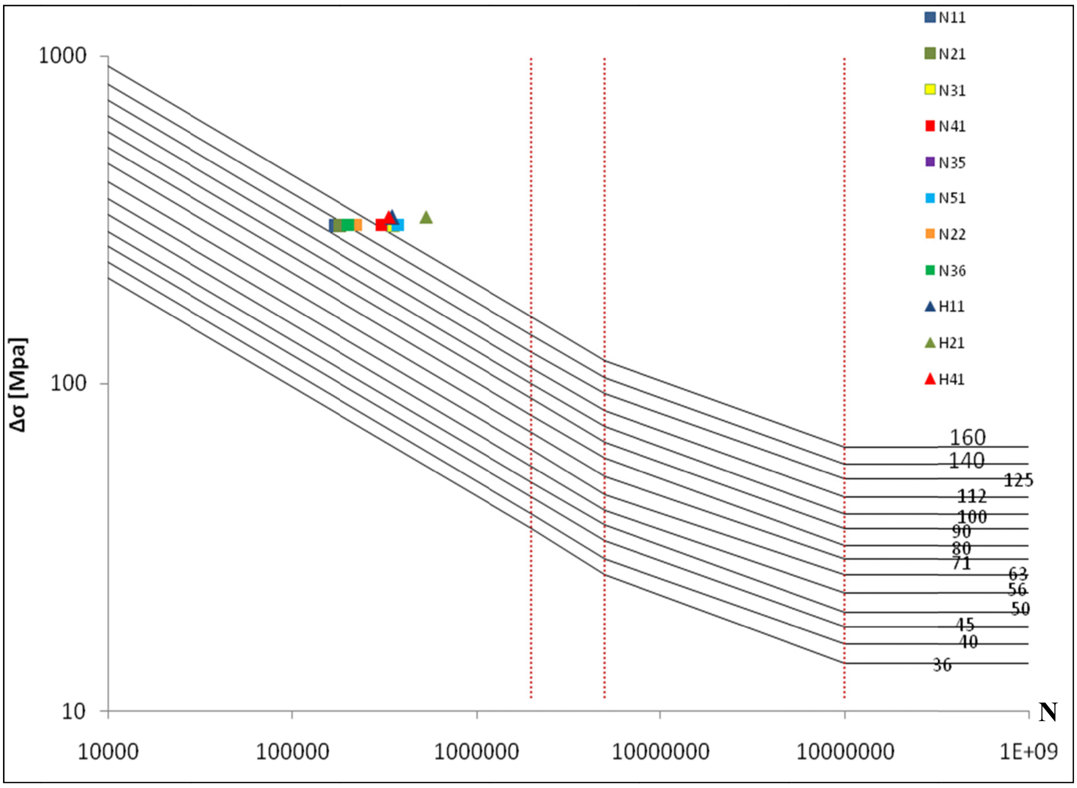


Figure 1. Fatigue failures of FRP reinforced steel plates subjected to tension forces

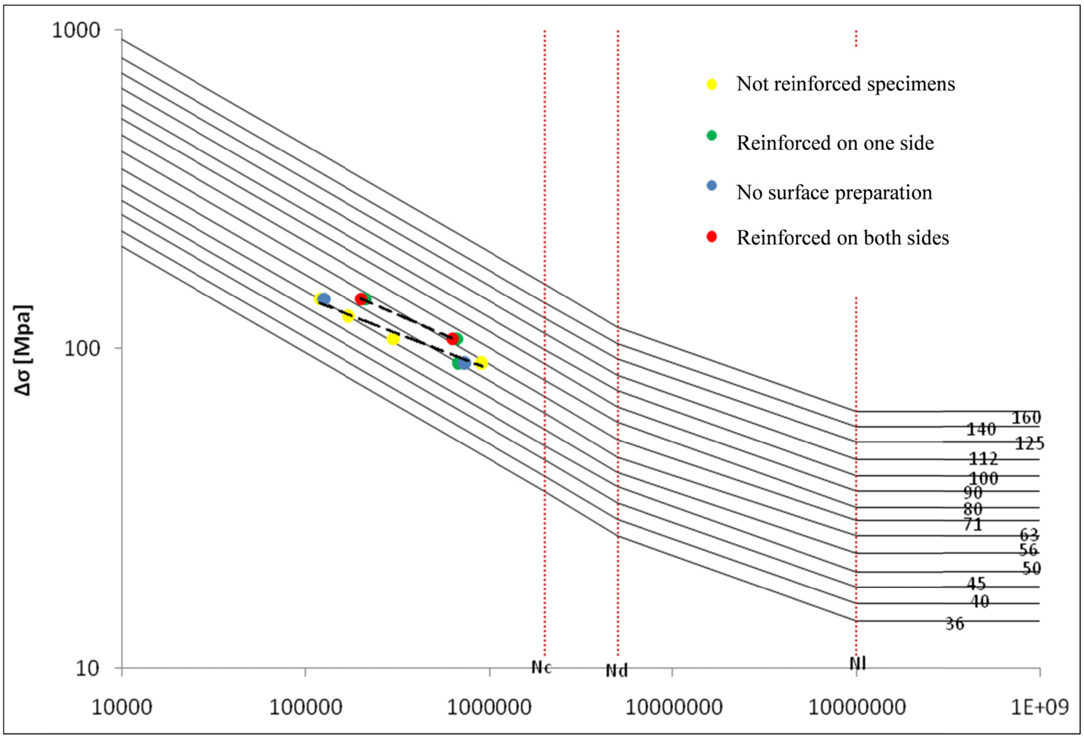


Figure 2. Fatigue failures and curve of FRP reinforced steel plates subjected to tension forces

Table 2. Bending test, FRP reinforced

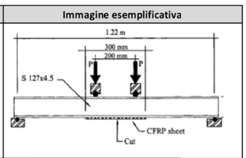
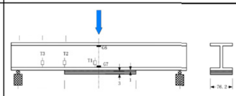
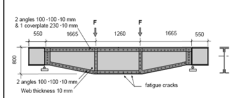
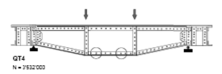
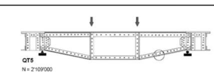
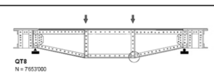
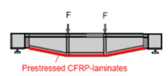
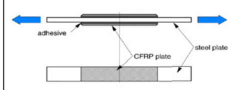
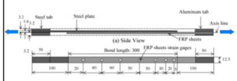
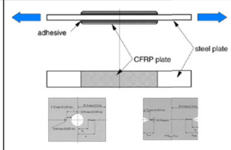
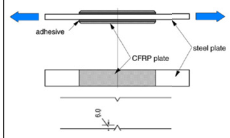
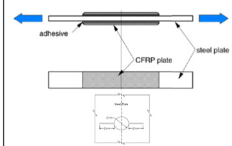
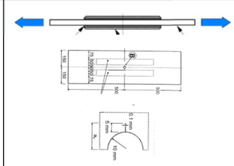
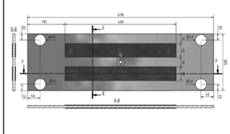
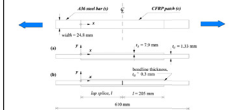
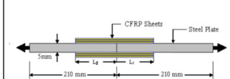
	Riferimento	Campioni	Tipo di test	Dim. materiali	Caratteristiche materiali	Dettaglio	Δσ [Mpa]	N	Note sulla rottura	Immagine esemplificativa
10	[Tavakkolizadeh 2003]	5 steel beams A36 notched on flanges and reinforced with CFRP on the lower side at the midspan	Bending test	Steel beam: S127x4.5, 1.22m long. CFRP: 300mmx76 mmx1.27 mm.	Fy=336.4 Mpa, 330.9 Mpa, E=194.4 GPa .CFRP: E=144 Gpa,shear strenght=2137 Mpa	Beam reinforced at the lower flange at midspan	207 241 276 310 345	1.191.140 71.278 35.710 30.216 19.068	Crack initiation and growth are similar in reinforced specimens, but quite different in the not-reinforced specimen. Crack initiate always in the edge of nothces going up to the beam panel.	
11	[Deng 2007]	UB13 steel beams CFRP reinforced on the lower flanges at midspan	Bending test	Steel beams: 127x76UB13, 1.2 m long. FRP: 3 mm thick, 400 mm long.	Steel E=205 Gpa. Sikadur 31 Normal, E = 8 Gpa, ft= 29.7 Mpa.	Beam reinforced at the lower flange at midspan	39 30 27	20.000 130.000 965.000	The crack propagates from the mid of the adhesive up to the steel-adhesive interface, with an angle of 45°.	
12	[Bassetti 1999]	Variable sections steel riveted beams reinforced with traditional tecniques.	Bending test	Beam: 5700mm long, angle profiles 100x100x10 mm, plates: 230x10 mm	Steel beam: E=212 Mpa, fu=412 Mpa, fy=312 Mpa.	QT7 one plate reinforced QT6 rivets replaced with HS bolts QT6 with stop holes QT6 with stop holes and HS bolt QT2 rivets replaced with bolt QT2 rivets replaced with bolt and a stop hole QT1 rivets replaced with bolt	80 72 91 72 71 71 91	5.918.000 12.177.000 16.644.000 17.185.000 6.217.000 7.391.000 6.144.000	In these beams the failures stands commonly on rivet holes. In case of short cracks, propagation could be stopped replacing rivets with HS bolts.	
13	[Bassetti 2001]	Variable sections steel riveted beams not reinforced.	Bending test	Beam: 5700mm long, angle profiles 100x100x10 mm, plates 230x10 mm.	Steel beam: E=212 Mpa, fu=412 Mpa, fy=312 Mpa.	QT4	80	3.532.000	The failure of the beam started from the rivet holes.	
						QT5	80	2.109.000	The failure of the beam started from the rivet holes.	
						QT8	80	7.653.000	The failure of the beam started from the rivet holes.	
		Variable sections steel riveted beam, with 2 stripes of CFRP normal strenghtening and 3 stripes of CFRP post-tensioned strenghtening	Bending test	Beam: 5700mm,angle profiles 100x100x10 mm, 230x10 mm plates. CFRP thickness: 1.4 mm	Steel beam: E=212 Mpa, fu=412 Mpa, fy=312 Mpa. CFRP SikaCarboDur M614.	QT4	80	20.229.000	QT4: the test has been stopped with no failures; QT5, QT8: crack propagation observed.	
						QT5	80	10.440.000		
						QT8	80	2.392.000		

Table 3. Tensile test on single steel plates, FRP reinforced

	Reference	Specimens	Test type	Dimensions	Materials	Details	Δσ [Mpa]	N	Failure notes	Image
1	[Bocciarelli 2009]	S275 specimens reinforced on both side with Sika carbodur M614	Tensile test	Steel specimens: 1000 mm long, 50 mm large, 6 mm thick. FRP:500x50x1.4mm.	Steel S275: E=21 GPa, nominal strenght 331 Mpa, minimal tension strenght 430 MPa.CFRP SikaCarboDur M614: E > 200 Gpa, ft > 2800 Mpa. Sikadur resin 30: E = 12840 Mpa, ft=30.2 Mpa	FT1 FT2 FT3 FT4 FT5 FT6 FT7	83 83 100 100 120 120 160	5.800.000 4.900.000 1.800.000 1.700.000 1.200.000 1.200.000 250.000	Failure mechanism started with the detachment from the edge of the element and then propagates on the whole element interface. A visual inspection reveals that the adhesive-metal interface is the weakest point of the reinforced system.	
2	[Iwashita 2007]	Single-lap shear steel plates	Tensile test	Steel specimens: 400 mm x 12.5mm x 1.6mm. FRP: nominal thickness=0.128mm	Steel SM490: tension strenght 490 Mpa, E=210 Gpa. FRP: tension strenght 3500 Mpa, E=235 Gpa. Epoxy resin: tension strenght 51.9 Mpa, E=3.53 Gpa	SF7 SF71 SF66 SF55 SF44	195 179 162 130 98	4.412 6.255 14.058 34.362 93.199	The most common failure mechanism has been found to be the detachment of the resin from the steel plates.	
3	[Jones 2003]	5 steel plates not reinforced and 24 steel plates reinforced on one or both side	Tensile test	Steel specimens: 6.5x51x510mm. SikaCarboDur thickness: 1.2mm; SikaWrapHex thickness 103C:1.0 mm; MBRACE CF130 thickness:1.0 mm	Steel fy=340 Mpa, fu=483MPa. E(SikaCarboDur)=165000MPa; E(SikaWrapHex 103C)=65000MPa; E(MBRACE CF130)=38000MPa	N11 not reinforced N21: 4 Sika Wrapex 103C layer N31-N34: Sika Wraphex 103C on two sides L=255mm N22: as N21 but with Mbrace CF130 N36: as N31 but with Mbrace CF130 H21-H22: Sika Wraphex 103C on both sides L=255mm H41-42: as H21 but with Sika Carpodur	305 305 305 305 305 323 323	173.418 182.829 353.328 222.731 200.851 533.772 336.455	Failure has been observed for CFRP delamination for all tests, no failure has been observed in the fibers, fatigue cracks propagates bejond CFRP fibers.	
4	[Monfared 2008]	15 steel plates notchedm with and without FRP (on both sides), sanded or not.	Tensile test	Steel specimen: 9.62mm x 50.8mm x 350 mm. Dim. FRP: 1mm x 38mm x 250 mm.	Steel: nominal strenght =350 Mpa, E = 200GPa. E(CFRP)=607GPa, tension failure CFRP =2160MPa. E(adhesive)=12.8GPa	UR-1 UR-2 UR-3 UR-4 R1S-S-1 R1S-S-2 R1S-S-4 R2S-S-2 R2S-S-3 R2S-S-4	90 108 126 144 90 108 144 108 126 144	907.533 302.500 172.325 118.192 672.584 663.784 211.336 625.000 33.491 199.251	Failure lies on the FRP for reinforced specimen , for delamination in not sanded specimens.	
5	[Zheng 2006]	6 steel specimens hollowed at the centre, with 2 indentation, reinforced with CFRP on one or both side.	Tensile test	Steel specimens: 100x10x700 mm. FRP: 100x300x1.0 mm, 100x300x1.4 mm.	Steel: fy=435 Mpa. FRP; En=165 Gpa, Eh= 320 Gpa. Aradilte adhesive shear E=2015 0.9 Gpa	-	120 90	1.000.000 3.312.000	The remaining fatigue life is extended from the 155 to the 580% for reinforced specimens. The propagation speed of cracks in reinforced specimens on both sides is lower than that of one side reinforcement. The strenght is higher with CFRP of high modulus and reinforced on both sides.	
6	[Colombi 2003]	Steel specimens hollowed at the centre with 2 cracks, representing the rivets holes, reinforced with 2 CFRP stripes post tensioned on both sides.	Tensile test	Steel specimens: 1000mmx300mmx10mm. SikaCarboDur stripes S512: 500mmx 50mmx1.2mm. Post tensioned stripes with high module: 500mmx50mmx1.4mm. Adhesive: 500x50x0.3mm.	Steel: E=210 Gpa. E(SikaCarboDur S512)=155 Gpa. E(SikaCarboDur M614)=210GPa. Adhesive E=714 Mpa	- not reinforced - reinf. on both side - reinf. on both side with post tensioned CFRP - reinf. on both side with post tensioned CFRP	80 80 80 80	300.000 900.000 1.800.000 6.000.000	A detachment on the interface adhesive-steel has been observed and the adhesive higher thickness has been demontrated to grow up the shear deformation on the adhesive layer.	
7	[Taljsten 2009]	5 metal specimens hollowed provided with two initial cracks	Tensile test	Steel specimens: 670x205mm. FRP: 400x50x1.4mm	Steel: Fy=294 Mpa, Ft=382 Mpa. CFRP E50C: Ft=2000MPa, E=155 Gpa. CFRP M50C: Ft=2500Mpa, E=260 Gpa. Adhesive BPE 567: shear strenght=52.5 Mpa, E=6.24 Gpa, G=2.4 Gpa. Adhesive Pox SK 41: shear strenght==81 Mpa, E=9.87 Gpa, G=3.8 Gpa	A1: not reinforced B1: E-CFRP S adhesive B2: E-CFRP S adhesive B3: E-CFRP S adhesive B4: E-CFRP S adhesive C1: B CFRP M adhesive D1: CFRP E post tensioned adhesive D3: CFRP E post tensioned adhesive	97,50 97,50 97,50 97,50 97,50 97,50 97,50 97,50	470.000 1.150.000 1.470.000 1.510.000 1.250.000 1.760.000 3.780.000 4.930.000	The detachment lies on the interface steel-FRP.	
	Riferimento	**Campioni**	**Tipo di test**	**Dim. materiali**	**Caratteristiche materiali**	**Dettaglio**	**Δσ [Mpa]**	**N**	**Note sulla rottura**	**Immagine esemplificativa**
8	[Matta 2005]	Steel bars A36, single jointed (SDS) or double jointed (DDS), reinforced with FRP on both sides.	Tensile test	SDS: 610 mm long, 24.8 mm large, 7.9 mm thick. DDS: 410 mm long, 1.33 mm thick.	Steel: Fy=305.7 MPa, Fu=445.2 MPa and E=196 Gpa. Carbodur ultimate strenght=2,990 MPa, longitudinal module=165.7 Gpa. Sikadur adhesive 30 tensile strenght=24.8 MPa, E=4.5 Gpa.	SDS DDS	83 73	1.000.000 1.000.000	Provini confrontati con categorie di dettagli saldati 36* e 50* EC3. SDS: distacco dell'interfaccia acciaio/adesivo. DDS: la rottura è una combinazione di un modo coesivo nell'adesivo e il distacco matrice/fibra dentro il composito.	
9	[Liu 2005]	12 plates jointed at the centre reinforced with CFRP on both side, normal and high module	Tensile test	Steel specimens: 50 mm large, 5 mm thick, 210 mm long. FRP: 80+60mm, 80+40mm	Provini d'acciaio: E=195 Gpa, ft = 484 Mpa. Mbrace CF 130: E = 240 Gpa, ft = 3800 Mpa. Mbrace CF 530 : E = 640 Gpa, ft = 2650 Mpa.	N40-1 N60-1 H40-1 H60-1	90 55 95 64	800.000 1.000.000 523.000 1.500.000	ID = failure to detachment in the interface. FB = fiber failure.	

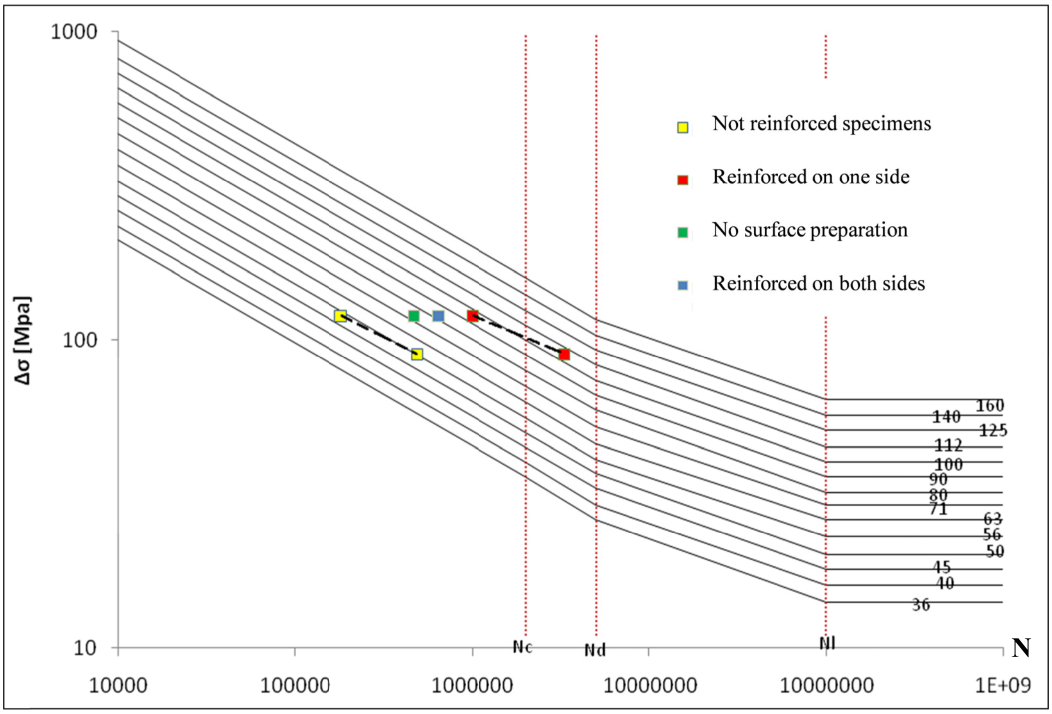


Figure 3. Fatigue failures and curve of FRP reinforced steel plates subjected to tension forces

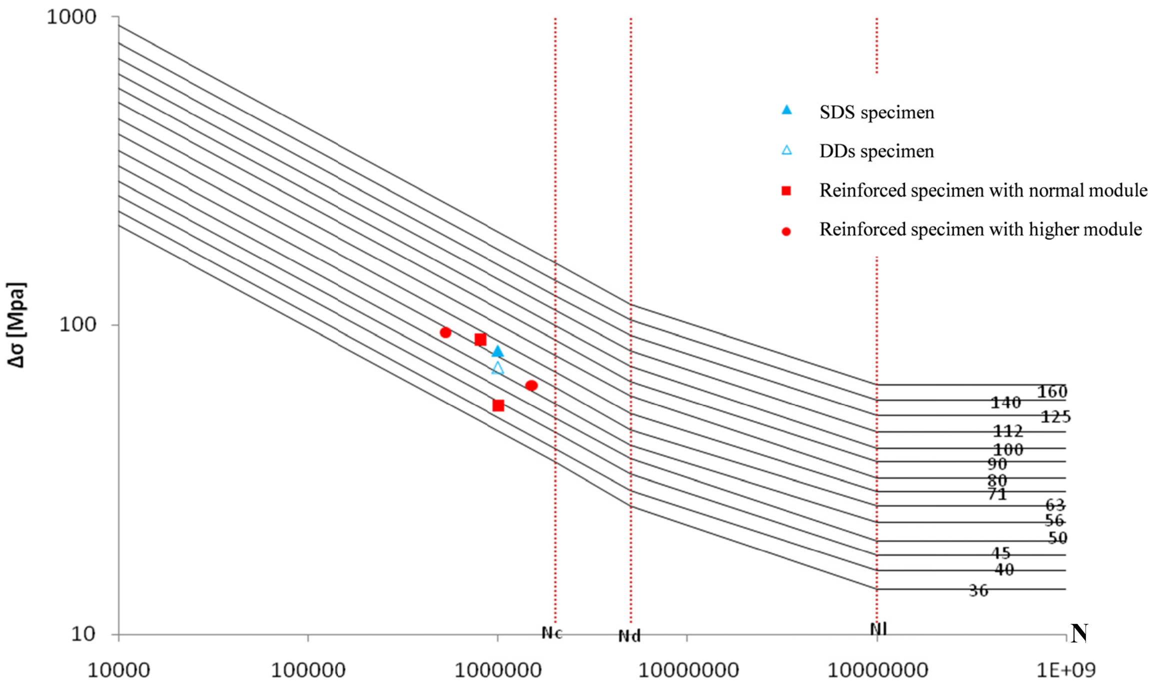


Figure 4. Fatigue failures of double strap FRP reinforced steel plates subjected to tension forces

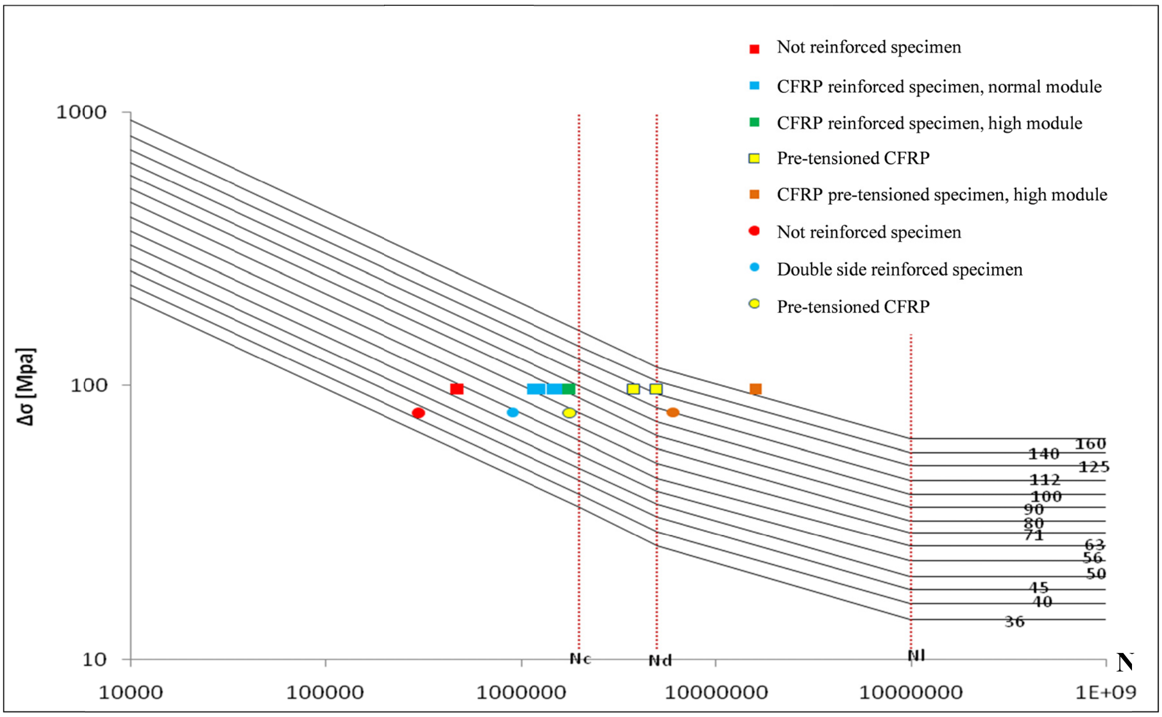


Figure 5. Fatigue failures of existing plates with FRP reinforcement subjected to tension forces

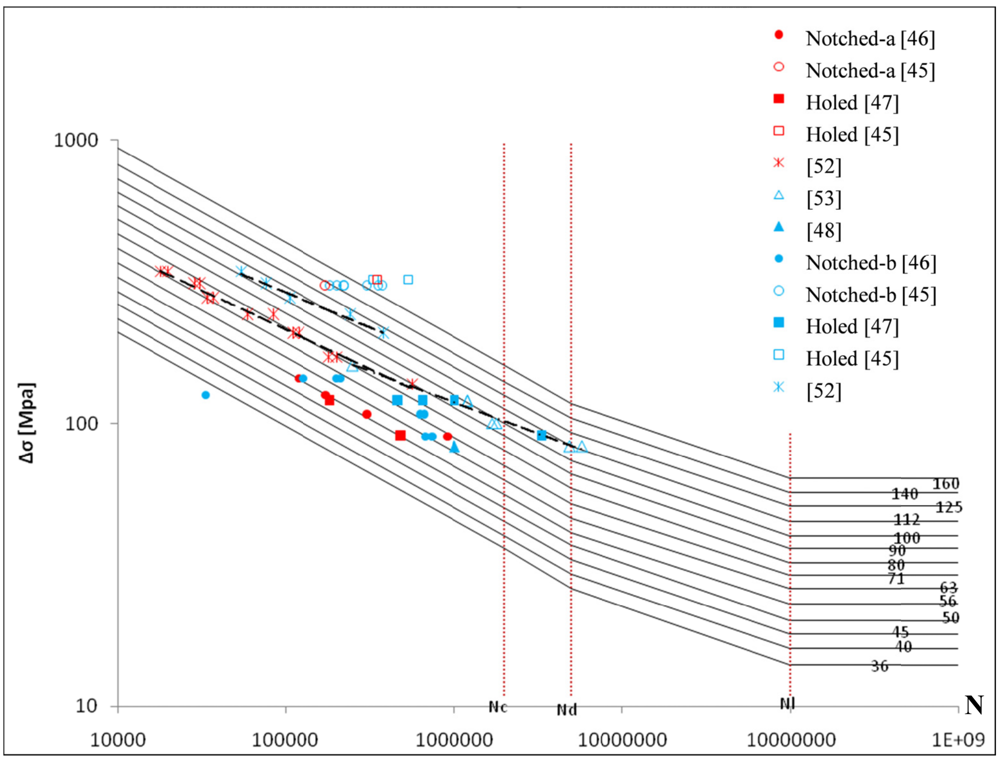


Figure 6. Fatigue failures and curves of different tests

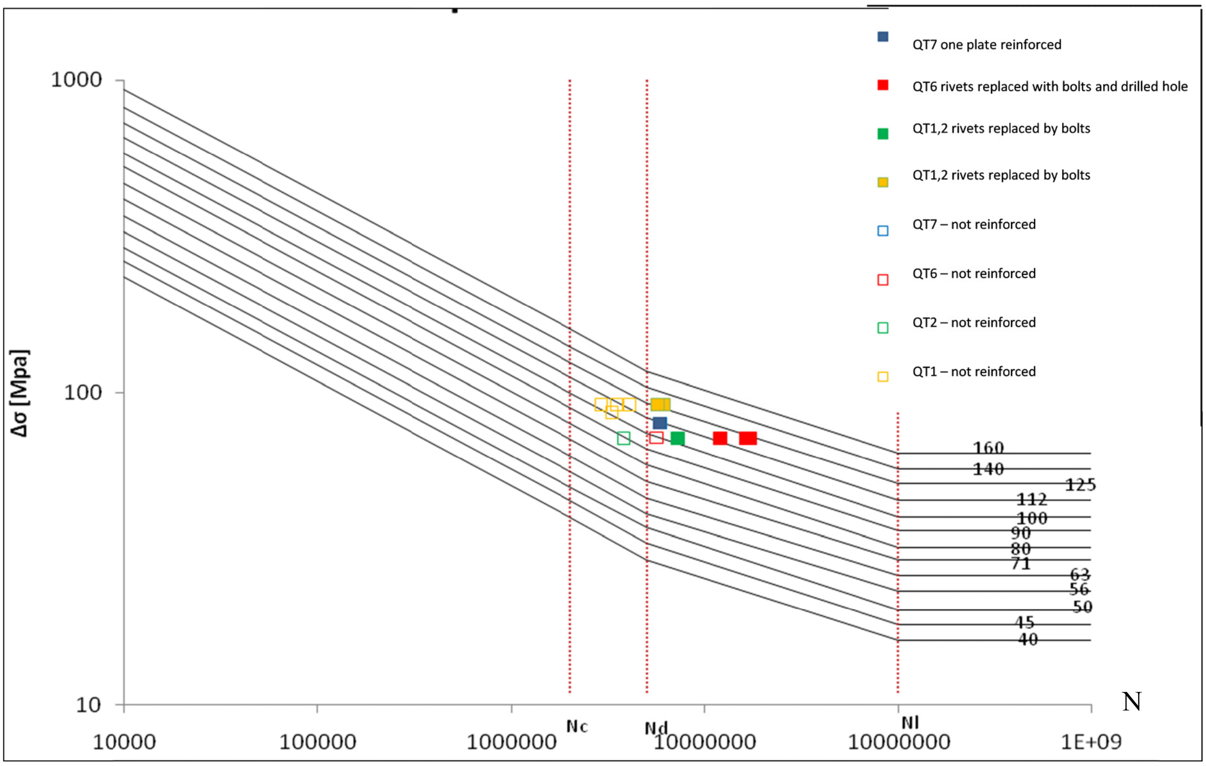


Figure 7. Fatigue failures of steel beam with traditional tecniques subjected to bending

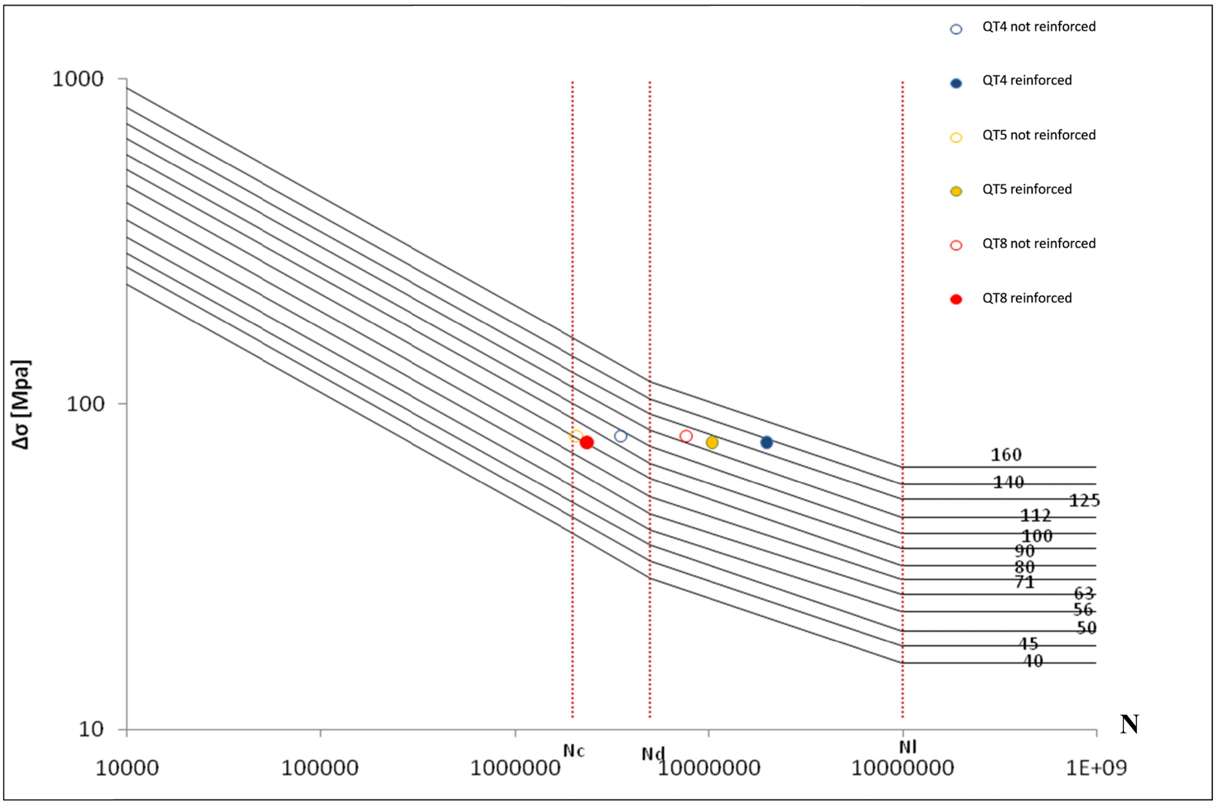


Figure 8. Fatigue failures of steel beam with CFRP reinforcing subjected to bending

5. Conclusions

This study deals with the analysis and comprehension of the fatigue behaviour of steel bridge joint, reinforced with FRP materials. The actual state of art of FRP strengthening is presented and discussed in the first part of this paper, while in the second part experimental tests coming from literature are presented and analyzed. It should be noted that, although a sufficient number of experimental tests have been found in literature, FRP reinforcing experimentations are more frequent for reinforced concrete than for steel structures, because up to now steel engineering have privileged steel to steel interventions rather than FRP rehabilitations. The use of FRP offers advantages compared to traditional techniques, because the FRP has a high strength / weight ratio, higher than that of steel; moreover they are very resistant against corrosion, if not entirely free from this, and extremely manageable. However, they are not sufficiently resistant to fire: FRP loses its strength and stiffness properties with temperature and the degradation in FRP properties is faster as compared to concrete or steel since the properties of the FRP matrix start to deteriorate even at modest temperature; also, bond degradation is another concern with externally bonded FRP. The analysis of experimental research found in the literature for the fatigue behavior of FRP reinforced steel detailing has proven the effectiveness of the composite material to extend the fatigue life of steel structures and decrease the growth of cracks. Two main types of fatigue testing has been analyzed, tensile and bending tests. Moreover a distinction has been made for test on individual steel plates reinforced with FRP, test on two samples of steel combined with FRP on both sides and the third is the tests on beams reinforced with FRP at the bottom. Failures data and their correlation were obtained from experimental data and compared to SN curves. Details reinforced with FRP belong to categories higher than those mentioned in EC3: in fact, for e.g., the graph of the fatigue curves of small-scale details of specimens reinforced virgin belong to the category 102, the notched specimens at 75, drilled to test 109, the old metal reinforced with 90 and those pre-tensioned to 125. From this analysis were then derived for the design of appropriate observations. The results obtained have shown a generally better fatigue behavior of steel reinforced detail than non-reinforced. Concerning the categories of Eurocode 3-1-9, reinforced details improved the ductility of the joint and demonstrated an higher life if compared to those of welded details, and it was also noted that the fatigue life of the steel specimens that have drilled holes is extended if compared to that of steel specimens which have cracks. This could be confirmed by an analysis of linear elastic fracture mechanics: the propagation of a crack can be described by a law of Paris as known, which takes into account the amplitudes of the effective stress intensity factors, which confirms that the speed of propagation of the crack will be higher and the fatigue life minor. The type of failure in almost all cases appears to be the detachment of the CFRP from the steel surface, without damaging the composite. This demonstrates the high capacity of resistance of fiber-reinforced composite and the weakest point of the system is the interface adhesive/steel. Finally it was found that the failure modes depend on the modulus of elasticity of the CFRP, the type of adhesive and the thickness of the adhesive.

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