Strengthening of Steel Connections in a 92-Year Old Railway Bridge using Prestressed CFRP Rods: a Multiaxial Fatigue Design Criterion

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Abstract

This paper presents a new retrofit system for strengthening the stringer-to-floor-beam double-
angle connections in a 92-year old riveted railway bridge in Switzerland, using prestressed
carbon fibre-reinforced polymer (CFRP) rods. The strengthening system transmits the forces
purely through friction, with minimum interference with bridge traffic. The system consists of
two components: a newly developed mechanical wedge-barrel anchor to hold the prestressed
CFRP rod, and a clamping system attaching the stringer flange. The strengthening system

reduces the out-of-plane deformation of the connections, resulting in a reduction in the distortion-induced stresses in the connections. The short-term bridge measurements showed that the application of a total prestressing force of 100 kN reduced the dominant mean stress in the connection hotspot by 47% (from 22.9 to 10.9 MPa) under the passage of passenger trains, while the stress range remained unchanged. Using the modified Wöhler curve method (MWCM) as a critical-plane-based multiaxial fatigue model, the strengthening system was observed to reduce the multiaxial mean-stress parameter by 30% under the passenger train loads. The long-term monitoring of the strengthening system, using a wireless sensor network (WSN) system, indicates no prestress loss in the CFRP rods during the seven-month period since its installation.

Keywords: Carbon fibre-reinforced polymer (CFRP) rod; Multiaxial fatigue; Post-tensioning; Strengthening bridge connections; Wedge-barrel anchor.

Introduction

In Europe, nearly 70% of all steel bridges are more than 50 years old, and 30% are over 100 years old (Bien et al. 2007). In the US, more than 29,000 roadway steel bridges are classified as structurally deficient (FHWA 2016). The Engineers Australia Report Card 2000 (IEAust 1999) has provided bridges with a rating of D (poor) and stated that an increase in the mass limit will exacerbate the situation. A similar situation exists in Japan (Yamada et al. 2002). Therefore, the problem of bridge aging is worldwide. Bridge replacement is costly; the US Federal Highway Administration has estimated that in 2016, the total cost for the replacement of all structurally deficient highway bridges is more than 47 billion dollars. It is estimated that the cost for bridge rehabilitation is 68% of the cost for their complete replacement (FHWA 2016). Therefore, development of reliable rehabilitation techniques has a significant economic impact.

Aging steel bridges suffer globally from fatigue problems. Fatigue cracks can initiate in different details of steel bridges. In general, bridge members are subjected to two different types of fatigue loadings: load-induced and distortion-induced fatigue loading. For the first type, design codes and standards normally provide formulations to determine the allowable fatigue load limit and to prevent the fatigue cracking. However, although the modern specifications provide some prescriptive rules to address the distortion-induced fatigue in new bridges through improved detailing, thousands of older bridges are susceptible to such cracking (Connor and Fisher 2006). Previous works have reported that around 90% of fatigue damage in steel bridges was caused by distortion-induced loading (Connor and Fisher 2006; Haghani et al. 2012).

According to different studies (Al-Emrani 2006; Haghani et al. 2012), one of the most common types of distortion-induced damage occurs in the stringer-to-floor-beam angle connections, i.e., more than 16% of all the fatigue damage cases collected in (Haghani et al. 2012) and the most frequent case. This type of connections is a common detail in many existing steel bridges. A number of fatigue-crack cases have been reported for this type of connection (Al-Emrani 2002; Al-Emrani et al. 2004). In most cases, fatigue cracks appeared in the angle fillet. In other cases, rivet failure was observed, where the fatigue crack began in the junction between the shank and the head of the rivets. In this case, the insufficient clamping force (Leonetti et al. 2020) causes bending as well as a prying effect, leading to the rivet head popping out (Al-Emrani 2002). These two fatigue failure types in connections are shown in Fig. 1.

As shown in Figs. 1(a) to (d), the two mechanisms causing distortion-induced fatigue cracks in these connections (Wilson and Coombe 1939; Haghani et al. 2012) are:

- Rotation of the stringer ends associated with bending:

Although double-angle connections are normally designed as simple shear connections, they have a certain amount of rotational stiffness; thus, they partially restrain the end rotation of the stringer, inducing tensile stresses in the connections (see Figs. 1(b) and

(c)). Depending on the relative stiffness between the stringer and the floor beam, either top or bottom of the connections could be in tension. The latter occurs because of the relatively low stiffness (i.e., large deformation) of the floor beam compared to that of the stringers, as shown in Fig. 1(c). Conversely, when the floor beam is stiff, the negative bending in the connection results in tension in the top of the connections, as demonstrated in Fig. 1(b). These tensile stresses have a relatively short influence line, and therefore, a significant number of cycles (e.g., in the order of axle load numbers) are applied by the trains passing over the bridge.

- Overall interaction between the floor system and the main trusses:

In through-truss bridges, the vertical load of axles is transmitted to the stringers through sleepers. The floor beams carry the vertical reactions of the stringers and transmit them to the main truss. The global flexural deformation of the main truss bridge involves the longitudinal displacement of the truss joints, δ , to which the floor beams are connected (see Fig. 1(a)). However, the floor beams are partially restrained from following this deformation because of the axial stiffness of the stringers and their connections to the floor beams. As a result, the additional axial forces, T , develop in the stringers and their connections to the floor beams. In this case, the floor beams are subjected to additional bending around the weak axis, as depicted in Fig. 1(d). This type of interaction has an influence line that is equal to the bridge span, and, in the case of railway bridges, there is essentially one loading cycle owing to the passage of each train. This phenomenon was first explained by Wilson and Coombe 1939, and later, was verified through onsite bridge testing by Al-Emrani 2002; Al-Emrani et al. 2004.

In order to prevent fatigue cracks in the angles, the design formulations have been proposed by the American Railway Engineering and Maintenance-of-way Association (AREMA 2016) as well as Fisher et al. 1987; Fisher 1990, to ensure that the angles have enough flexibility.

98 However, many existing bridges do not satisfy the recommended formulations (Connor and
99 Lloyd 2017). Therefore, the repairing techniques have been proposed by different researchers
100 to solve the fatigue problem with existing bridges.

101 Traditional retrofitting techniques for retrofitting riveted metallic structures include the use
102 of high-strength pre-tensioned bolts (Reemsnyder 1975; Baker and Kulak 1985; Al-Emrani
103 2002), welding additional elements, stop holes, repair of support conditions, etc. (Kuehn et al.
104 2008; Dexter and Ocel 2013; Connor and Lloyd 2017). Additionally, in some cases, the
105 connections were softened by removing some rivets (Bowman 2012). The stiffness reduction
106 in the connection, however, resulted in larger deformations, and led to some damage in other
107 elements. These strengthening solutions could not permanently solve the fatigue problem in the
108 connections and led to re-initiation of the fatigue cracks either from the same location or from
109 another location (Roeder et al. 2005). For example, in the case of stringer-to-floor-beam angle
110 connections, replacing the rivets with pre-tensioned bolts reduces the risk of fatigue failure in
111 the fasteners but it does not solve the fatigue problem in the angle fillet, as it does not reduce
112 the out-of-plane deformations.

113 Recently, carbon fibre-reinforced polymer (CFRP) materials have been used for
114 strengthening metallic structures owing to their superior properties such as the higher fatigue
115 and corrosion resistance, and their light weight compared to steel (Zhao 2013). Both trapezoidal
116 (Ghafoori and Motavalli 2015) and flat (Hosseini et al. 2018) prestressed unbonded
117 reinforcement (PUR) systems were developed, in which prestressed CFRP plates were held
118 using mechanical clamps working purely through friction under a symmetric loading condition.
119 The systems were used for strengthening steel girders against flexural (Hosseini et al. 2018),
120 buckling (Ghafoori and Motavalli 2015), and fatigue (Ghafoori et al. 2015b) loads. In addition,
121 they were applied to strengthen the girders of an old steel bridge in Switzerland (Ghafoori et al.
122 2015a) and Australia (Ghafoori et al. 2018; Hosseini et al. 2019). Furthermore, prestressed

shape memory alloy (SMA) plates were used in a PUR system for strengthening stringer-to-floor-beam double angle connections (Izadi et al. 2019); however, this was only performed in the laboratory and not in any actual applications.

Development of a strengthening system being applicable to the real bridge connections is a greater challenge. Fig. 2-a shows the details of a stringer-to-floor-beam double angle connection including sleepers and rails in a typical riveted steel bridge. Owing to the complex geometry of such details (i.e., the presence of floor beams, sleepers, and rails), it is nearly impossible to use prestressed plates (CFRP or SMA) for strengthening connections. Therefore, existing retrofitting solutions are not applicable to such details.

Outline of the study

In this study, for the first time, an applicable retrofitting solution for bridge connections with such complex geometric details is introduced. In the proposed system, prestressed CFRP rods (rather than plates) are used and the results of static and fatigue tests on the proposed system are briefly described. Next, the installation of the system for strengthening connections in an old riveted railway bridge in Switzerland is presented. Then, to demonstrate the effectiveness of the system to reduce the stresses in the connection hotspot, the results of short-term on-site measurements are presented. Next, the long-term monitoring of the strengthening system performance using a wireless sensor network (WSN) system is described, and the results are discussed. Finally, a multiaxial design criterion based on the critical plane approach is presented to evaluate the effect of strengthening on the fatigue state of the connections.

Development of the strengthening system

The concept of the proposed system for strengthening the top of stringer-to-floor-beam double angle connections is depicted in Fig. 2. As shown in Fig. 2(b), the system consists of two key components: a wedge-barrel anchor and a clamping system. The wedge-barrel anchor consists of a steel shaft with a conical hole inside, and three split aluminium wedges being in

148 direct contact with the prestressed CFRP rod. The wedge-barrel anchor holds the prestressed
149 CFRP rod through friction. The clamping system consists of high-strength steel plates that are
150 connected together using high-strength bolts, as shown in Fig. 2(b).

151 By fastening the bolts in the bottom plates, the top and bottom plates grip the stringer flange,
152 providing frictional resistance. The CFRP prestressing force is transmitted to the clamping
153 system through the wedge-barrel anchor, and the clamping system transmits all the load to the
154 stringer through friction. Additional details regarding the dimensions and the design of the
155 strengthening system can be found in (Heydarinouri et al. 2020).

156 The prestressing force in the CFRP rod applies a compressive force, P , and a positive
157 bending moment, M_P , which counteracts with the negative bending moment in the connections
158 (see Fig. 2(a)). Thus, the out-of-plane deformation, and consequently, the distortion-induced
159 stresses in the connections are reduced. In the following sub-sections, a brief description of the
160 experimental results of the wedge-barrel anchors and the clamping is provided.

161 ***Laboratory tests on the retrofitting system***

162 To hold the prestressed CFRP rods, with a diameter of 8 mm, a new mechanical wedge-
163 barrel anchor was developed at the Structural Engineering Research Laboratory Empa,
164 Switzerland. The reliable performance of the wedge-barrel anchor was investigated through
165 static and fatigue tests according to the standard procedures proposed by *fib* 1993; Rostásy
166 1998; EOTA 2002. The test results showed that the anchor fulfils the requirements of the codes
167 and standards. The key results are summarised as follows:

168 In all static tests, the failure mode was the fracture of the CFRP rods, with an average tensile
169 strength of 2370 MPa, which was higher than the guaranteed tensile strength of the CFRP rods
170 (i.e., 2047 MPa). In addition, during the fatigue loading for 2 million cycles, neither slippage
171 nor the CFRP rod failure occurred.

The fatigue performance of the clamping system was also experimentally investigated. The fatigue tests were conducted in a set-up shown in Fig. 3, in which two IPB 400 stringers, each with a span of 2.66 m, were connected to an IPB 550 floor beam, with a span of 1 m, using four L160 × 160 × 15 web-to-web angle connections. In this set-up, the stringers on one side and the floor beams on both sides rested on rolling supports. The dimensions of the parts in the set-up were initially similar to those in the bridge, which was to be later strengthened. Fatigue loadings were applied on the stringers with two 500 kN vertical actuators. The load ratio in the fatigue tests was 0.1 with the maximum load of 240 kN per actuator. The prestressing of each CFRP rod in the fatigue tests was 995 MPa (equivalent to 50 kN).

The fatigue tests showed that after 11 million loading cycles, no failure occurred in the CFRP rods. Moreover, no prestressing loss was observed in the CFRP rods, implying that during the fatigue tests, no slippage occurred in either the clamping systems or in the wedge-barrel anchors. The strengthening system reduced the stresses in the connection hotspots by more than 40%. It is noted that the connection hotspot, which is located on the angle fillet, were determined based on the finite element (FE) model, as described in (Heydarinouri et al. 2020).

Effect of removing the bolts

To install the strengthening system, two bolts/rivets (from the second row of the fasteners) have to be removed from the angles to let the CFRP rods pass the floor beam (see Fig. 2 (a)). As shown in Fig. 3, the removed bolts were replaced with two hollow bolts. The strains in the angle hotspot were measured during bolt removal. In the connection hotspot, a rosette strain gauge with directions a, b, and c (a and c were perpendicular to each other) was applied to measure the state of stress in the connections. During bolt removal, the strain change in different directions was negligible, i.e., less than 10 microstrains.

Furthermore, before the installation of the strengthening system, the external loads on the stringers were statically increased up to 240 kN (as the maximum load in the fatigue tests) for

the connections with and without bolts. The removal of the bolts did not change the strain in different directions during the application of the external loads. This observation is consistent with existing experimental results (Munse and Petersen 1959; Kulak et al. 2001), indicating that the removal of the bolts/rivets from the second row of bolts does not affect the out-of-plane deformation of the connections.

Advantages of the proposed strengthening system

Considering the above-mentioned results and descriptions, the proposed strengthening has the following advantages:

- The proposed system is capable of lowering the fatigue failure risk in the angle fillet and rivets simultaneously by reducing the out-of-plane deformations. Particularly, considering the fact that there is a considerable variation in the rivet clamping forces (Leonetti et al. 2020), the proposed prestressed system is a reliable alternative to previous retrofitting methods such as replacing every single rivet with pre-tensioned bolts.
- Unlike the previous PUR systems, the proposed clamping system is reliably performing under asymmetric loading conditions, increasing the versatility for different structural details and applications.
- The use of the CFRP rods rather than plates makes it possible to pass the rod through the hole. Therefore, it can be used for complex geometries such as stringer-to-floor-beam connections.
- As for the installation of the strengthening system the space under the bridge deck is used, it has minimum interference with bridge traffic. In addition, the proposed system minimises the structural damage, i.e., it works solely through friction, without the need to drill a hole in the parent structure.

- The applications of the developed wedge-barrel anchor in the civil engineering domain are not limited to the strengthening of metallic connections. The system can be used as an alternative to conventional prestressed steel strands, e.g., in cable-stayed bridges, prestressed concrete decks and box girders, and post-tensioned walls (Sayed-Ahmed et al. 1999). It is in particular suitable when there are concerns related to the long-term durability of prestressed steel strands in terms of corrosion, fatigue and relaxation.

Geometrical details of Aabach Bridge

The Aabach Bridge (see Fig. 4) is an old steel railway bridge, which was built in 1928 on the main line between Zürich and Chur in Switzerland. It was originally designed as a riveted single-track bridge that contains 10 panels. The bridge was constructed over a river with a total length of 38.7 m, a width of 5 m, and a height of 4.6 m, as shown in Figs. 5(a) and (b). The Aabach Bridge was constructed on a skew such that it spans the river at a 35° angle. The bridge is subjected to both passenger and freight trains on a daily basis.

All elements of the Aabach Bridge are made of structural steel. The bridge girders are either rolled or riveted built-up. The top and bottom chords, diagonals, and vertical members are shown in Fig. 4. The two rails are mounted on wooden sleepers, and the wooden sleepers are bolted to the longitudinal stringers using steel angles. The double angle stringer-to-floor-beam connections are shown in Fig. 5(c). The stringers and floor beams have rolled IPB 400 and IPB 800 sections, respectively. The connection that was selected for the strengthening is shown inside the red circle in Fig. 5(a). The stresses in the left and right angles at this connection (see Fig. 5(c)) are investigated later in this study.

It is noted that the stiffness ratio (i.e., the ratio of the flexural stiffness of the stringer to that of the floor beam), γ , introduced by Al-Emrani 2002 is equal to 0.01 for the Aabach Bridge. The reason for such a small relative stiffness is that the floor beam, with a depth of two times greater than that of the stringer, has a greater flexural stiffness than the stringers. The latter

indicates that the tensile stresses develop at the top of the connections, as shown in Fig. 1(b), and, therefore, the strengthening system has to be installed on the top flange of the stringer.

Installation of the strengthening system on the Aabach Bridge

In this section, the stepwise installation procedure of the proposed strengthening system on the Aabach Bridge is described. Before the installation, two rivets were removed from the second row of the rivets in the angles located in either side of the stringer.

Presetting procedure

Once the two rivets were removed, the hollow bolts substituted the rivets, and the CFRP rods passed through the hole, as shown in Fig. 6(a). The diameter of the rivet hole was 22 mm and the hollow bolt was an M20, 8.8 with a hole diameter of 14 mm. The torque applied to fasten the hollow bolt was 100 Nm resulting in a pre-tensioning of 32 kN.

It is noted that the diameter of the hole has to be relatively larger than that of the rod in order to ensure that the rod does not touch the edge of the hole to prevent any transverse stresses on the rod and possible damaging the fibres. The CFRP rod did not touch the edge of the hole either in the laboratory tests or in the field application at the Aabach Bridge because the relative deflection of the CFRP rod inside the hole was very small. However, for the cases where there is a risk that the rod touches the edges of the hole, either the diameter of the hole should be slightly increased or a soft material could be placed around the rod inside the hole to prevent any damage to the carbon fibers.

Next, the clamping system was installed. On one side of the CFRP rods, the wedge-barrel anchors were already installed and preset, and on the other side, the wedge-barrel anchors were installed after the CFRP rods passed through the hollow bolt, as shown in Fig. 6(b).

To apply the presetting force to the wedges that were installed on-site, a presetting system shown in Fig. 6(c) was used. This system includes a ring with a threaded hole inside, which is

connected to the threaded area around the barrel. A presetting hollow shaft with a certain length is then placed around the wedges and on the top surface of the barrel. Then, a presetting top plate is placed on the wedges. The length of the presetting hollow shaft was determined based on the laboratory tests to provide an adequate presetting force. Six M8 bolts are passed through the holes in the presetting top plate, and then, the threaded holes in the ring. By fastening the bolts (see Fig. 6(c)), the presetting top plate pushed the wedges into the barrel. The fastening of the bolts continued until the top plate touched the presetting hollow shaft. To prevent the wedge-barrel anchor from rotating during the fastening of the six M8 bolts, two temporary presetting walls. Once the presetting procedure for both anchors was completed, the required parts for presetting, i.e., the presetting ring, the hollow shaft, and the top plate, were removed.

Procedure of CFRP prestressing

After presetting, each CFRP rod, with a diameter of 8 mm, was pulled with a prestressing force of up to 50 kN (equivalent to 995 MPa and 49% of the CFRP guaranteed tensile strength of 2047 MPa). The CFRP rods were manufactured by the S&P Clever Reinforcement Company AG, Switzerland,. The fibre volume fraction of the CFRP rods and the elastic modulus in the fibre direction were 65% and 160 GPa, respectively.

The prestressing force was applied to the CFRP rods by pumping two hydraulic cylinders, as shown in Fig. 7. As given in the figure, a temporary housing was connected to the clamping system acting as a support for the hydraulic cylinders. The parts that were previously used for presetting were used here as a connector between the wedge-barrel anchor and the hydraulic cylinder using a high strength M13 threaded rod, with a tensile capacity of 150 kN.

During the pulling, the prestressing level was monitored by two strain gauges applied on each rod. Once the desired value of the prestressing force was achieved, the oil pressure behind the cylinders was fixed, and then, the threaded hollow shaft (see Fig. 2(b)), was fastened until

it touched the back of the barrel. At this stage, the system became fixed, and, the oil pressure in the hydraulic cylinders was released.

The applied prestressing to the CFRP rods on the right and left sides of the stringer is illustrated in Fig. 8. As shown in Fig. 8, the prestressing level in both CFRP rods was approximately the same. In Fig. 8, a fluctuation in the prestressing level of approximately 1000 MPa is observed. The first reduction in Fig. 8 occurred after the fixation of the system using the threaded hollow shaft, resulting in a prestressing level lower than the target value. To compensate this prestress reduction, the CFRP rods were again pulled up to a force a bit greater than the target prestressing level. In this manner, after the second prestressing reduction, the required prestressing force in the CFRP rods was achieved.

Short-term measurements

To evaluate the strengthening effect, short-term measurements were conducted using a data acquisition system (HBM, AG, Darmstadt, Hesse, Germany), during the passage of different trains, as shown in Fig. 7. The strain gauges used for the short-term measurements are shown in Fig. 9. As shown in Fig. 9(a), two rosette strain gauges of type RY8x-3/120 (HBM, AG, Darmstadt, Hesse, Germany), with the grids in three different directions, were applied on the left and right angle fillets to obtain the multiaxial state of stress in the connection hotspots.

Furthermore, the strain in each CFRP rod was measured using strain gauges of type 1-LY66-6/120 (HBM, AG, Darmstadt, Hesse, Germany), shown in Fig. 9-b. these strain gauges were also used during the prestressing process. In addition, two unidirectional strain gauges of type 1-LV41-3/120 (HBM, AG, Darmstadt, Hesse, Germany), were used in the outermost top and bottom of the stringer web immediately after the stringer fillet ends. The total duration of the short-term measurements was four hours, i.e., two hours before and two hours after strengthening. The frequency of the measurement was 2400 Hz. The results of short-term measurements in the angles and the web are presented in the following sections.

319 *Left angle*

320 The location and the grid directions of the rosette strain gauge on the left angle are
 321 schematically illustrated in Fig. 10(a). The strain gauge was positioned 52 mm away from the
 322 top surface of the angle. The strain history in different directions measured by the rosette strain
 323 gauge on the left angle (see Fig. 5(c)) before the installation of the strengthening system is
 324 shown in Fig. 11(a). As shown in the figure, during the two-hour measurements, 17 trains
 325 passed over the bridge. The majority of the fluctuations were due to the passenger trains,
 326 resulting in nearly the same stress range. It is noted that to distinguish between the different
 327 curves, the high-frequency measurement noise was suppressed by means of a digital 50th-order
 328 lowpass filter with a cut-off frequency of 70 Hz in the MATLAB software.

329 The strain cycles for a passenger train, i.e., Train (6), is shown in Fig. 11(b). As shown in
 330 the figure, the strains in the a- and c-directions are approximately the same, while the strain in
 331 the b-direction has the greatest magnitude. From the measured values of the strains in the a-, b-
 332 , and c-directions, the principal strains in the x'-y' plane, i.e., ε_P and ε_Q , as well as its angle
 333 with the x' and y' axes, θ , were calculated with Eq. 1,

$$\varepsilon_P = \frac{\varepsilon_a + \varepsilon_c}{2} + \frac{1}{\sqrt{2}} \sqrt{(\varepsilon_a - \varepsilon_b)^2 + (\varepsilon_b - \varepsilon_c)^2} \quad (1.1)$$

$$\varepsilon_Q = \frac{\varepsilon_a + \varepsilon_c}{2} - \frac{1}{\sqrt{2}} \sqrt{(\varepsilon_a - \varepsilon_b)^2 + (\varepsilon_b - \varepsilon_c)^2} \quad (1.2)$$

$$\theta = \frac{1}{2} \tan^{-1} \left(\frac{\varepsilon_a - 2\varepsilon_b + \varepsilon_c}{\varepsilon_a - \varepsilon_c} \right) \quad (1.3)$$

334 Both the principal strains and stresses are in the P- and Q-directions. The principal stresses are
 335 calculated using the following equations:

$$\sigma_P = \frac{E}{1-\nu^2} (\varepsilon_P + \nu \varepsilon_Q) \quad (2.1)$$

$$\sigma_Q = \frac{E}{1-\nu^2} (\nu \varepsilon_P + \varepsilon_Q) \quad (2.2)$$

Where E and ν are elastic modulus and the poisson's ratio of the steel, respectively. According to the Mohr's circle shown in Fig. 10-b, to obtain the stresses in the x-y plane, they have to be calculated on the plane with an angle of $90-2\theta$ with respect to the principal directions P-Q in the Mohr's circle (i.e. angle of $45-\theta$ in the element). Therefore, considering plane stress conditions, the stresses in the x-y coordinate system are calculated as follows:

$$\sigma_x = \frac{\sigma_P + \sigma_Q}{2} + \frac{\sigma_P - \sigma_Q}{2} \sin 2\theta \quad (3.1)$$

$$\sigma_y = \frac{\sigma_P + \sigma_Q}{2} - \frac{\sigma_P - \sigma_Q}{2} \sin 2\theta \quad (3.2)$$

$$\tau_{xy} = \frac{\sigma_P - \sigma_Q}{2} \cos 2\theta \quad (3.3)$$

As the stress state on the surface is considered, the other stress matrix components in the x-y-z coordinate system are zero, i.e., $\sigma_z = \tau_{xz} = \tau_{yz} = 0$. Using $E = 200$ GPa and $\nu = 0.3$, the above equations were solved in the MATLAB to obtain the stress history in the rosette strain gauge located on the left angle before and after strengthening, and the results are depicted in Fig. 12. Before strengthening, as shown in Fig. 12-a, the stresses in the y-direction, σ_y , are generally less than one-third of those in the x-direction, σ_x . The shear stress, τ_{xy} , is negligible. This shows that the x and y directions are very close to the principal directions in the element. Additionally, in most fluctuations, the maximum stress in the x-direction is in the order of 40 MPa. These fluctuations were due to the passage of the passenger trains.

After strengthening, short-term measurements, with the same 2400 Hz frequency were conducted. During this period, 15 trains passed over the bridge. As shown in Fig. 12-b, due to the prestressing, a compressive stress of 12 MPa and 16 MPa was measured in the x- and y- directions, respectively, with the shear stress being approximately zero. Due to the passage of the passenger trains, most fluctuations have a nearly identical maximum stress in the x- direction, i.e., approximately 20 MPa.

It is noted that the short-term measurements were conducted as follows: at the end of the first 2-hour short-term measurements (before strengthening), the measurement was stopped. Then, it was resumed during the prestressing, as shown in Fig. 8. When the prestressing process was completed, the measurement was stopped again, and immediately, the next 2-hour short-term measurements (after strengthening) were conducted. Therefore, the time 0 s in Fig. 12-b corresponds to the start of the data recording when the prestressing process was completed.

The stress cycles before and after strengthening due to the passage of passenger trains (6) and (23) are depicted in Fig. 13. Note that the cycles in the figure did not occur simultaneously, i.e., they are depicted here together for the sake of comparison. Due to the passage of Train (6), a stress range of 36.2 MPa, with a minimum stress of 6.5 MPa is observed in the x-direction. After strengthening, the $\Delta\sigma_x$ stress range due to the passage of Train (23) is approximately 36.5 MPa, with a minimum stress of -12 MPa. It shows that the strengthening system reduced the mean stress by 18.35 MPa (i.e., from 24.6 to 6.25 MPa). However, the stress range magnitude before and after the strengthening is approximately the same. The stress range values in the y- direction before and after strengthening were 10.4 and 12 MPa, with the corresponding minimum stress being approximately 1.6 and -16 MPa, respectively (see Fig. 13-b). Neglecting shear stress τ_{xy} , the stress cycles for σ_x and σ_y show that the strengthening system reduces only the mean stress, and not the stress range. The reason is that the proposed strengthening system does not add a considerable flexural stiffness to the connections.

In addition to the passenger trains, there were other types of trains such as Trains (14), (19), and (23), which generated completely different cyclic patterns compared to the passenger trains. The stress cycles due to these three trains are depicted in Fig. 14. It is noted that Train (14) passed over the unstrengthened connections and Trains (19) and (23) passed over the strengthened connections. For Train (19), first, a large cycle was applied, and then, after a number of smaller cycles, the bridge was subjected to a series of larger load cycles. Conversely, for Train (26), after a large cycle, with the largest measured amplitude among all the trains, the subsequent cycles are negligible.

To investigate the strengthening effect under the load of Train (14), consider the cycles in Fig. 14-a. As shown in the figure, for some cycles, the mean stress value in the x-direction, σ_{mx} , is equal to 53.25 MPa. Considering the fact that the strengthening system does not change the stress range, and it reduces only the mean stress by 12 MPa in the x-direction, it is inferred that if exactly the same train passed over the bridge after strengthening, the mean stress would be equal to 41.25 MPa, which is equivalent to a 23% reduction in the mean stress. In the same manner, for the first cycle of Train (26), the reduction of the mean stress value would be from 31.85 to 19.85 MPa, resulting in 38% reduction in the mean stress.

The above descriptions indicate that the effectiveness of the strengthening system in reducing the stresses due to the passage of each train depends on the type of the train. To evaluate the strengthening effect, all the stress cycles applied to the left angle were collected; the stress cycles due to the passage of each train in the x- and y-directions were identified by the stress range $\Delta\sigma$, the mean stress σ_m , and the corresponding number of cycles n . A total of 229 cycles were applied to the connection by 32 trains (including the trains before and after the strengthening), ignoring the small cycles with stress ranges below 10 MPa in the x-direction. As most of the trains were passenger trains (i.e., same type of trains), the stress components that represent all the passenger trains were obtained. Therefore, the stress cycles due to Trains

(14), (19), and (26) were removed from the pool of passenger trains. The effect of strengthening on the stress components due to non-passenger trains were considered as separate cases.

The information related to the stress cycles due to the passenger trains are summarised in Table 1. In this table, the cycles due to Trains (18)–(32), which were obtained after the strengthening, were accounted as the cycles that would be generated if the strengthening system were not installed. It means that the mean stress values in the table due to Trains (18)–(32) were obtained by ignoring the compressive stress due to the strengthening. For the stress range, however, the measured values were directly used as the stress range was unaffected by the strengthening system.

The average values for the stress range and the mean stress in the x- and y-directions, as well as the standard deviations (SD) are presented in Table 1. Previous studies Kuehn et al. 2008 showed that the stress range fractile from 50 to 75% makes the largest contribution to the fatigue damage sum in metallic bridges because the number of cycles with greater stress ranges (i.e., the fractile from 90 to 95%) is very limited. However, to the best of the authors' knowledge, such a recommendation has not been proposed for the mean stress.

In this study, the values for the stress range and the mean stress are given for the percentile of 95th, $S_{95\%}$, (i.e., those that are greater than 95% of all values) as a more conservative value compared to 50 to 75%. These values can be used to evaluate the strengthening effect. For example, the compressive stress of 12 MPa in the x-direction reduces the mean stress value of the 95th percentile by 47% from 22.9 to 10.9 MPa. The compressive stress of -16 in the y-direction would reduce the 95th percentile mean stress from 9.8 MPa to -6.2 MPa. Later in this study, a multiaxial fatigue analysis is performed to investigate the fatigue state in the angles before and after strengthening.

Right angle

The stress history in the right angle hotspot (see Fig. 5-c) before strengthening showed that the stress levels were lower than those in the left side, i.e., in the order of 20 MPa in the x-direction for the passenger trains. The reason is that due to the overall interaction between the floor system and the main trusses, a flexural deformation occurs in the floor beam around the weak axis, as depicted in Figs. 1(a) and d. As a result, a greater tensile stress is imposed to the angle that is located closer to the floor beam mid-span (here, it is the left angle). Therefore, the fatigue state in the left angle is more critical than that of the right angle.

Stresses in the web

In the short-term measurement, the stress history in the stringer web, before and after the strengthening, was measured. The measurement results showed that the stress levels due to the passage of the trains are generally low even before the strengthening, i.e., lower than 16 MPa. The strengthening system caused a compressive stress of approximately 8 MPa in both the top and bottom of the web. The stress history due to the passage of Trains (6) and (14), provided in Appendix I, showed that the tensile stress in the stringer, due to the global deformation of the bridge is 4 and 7.5 MPa, respectively. The global distortion-induced tensile stresses in the stringer due to the passage of all trains (including non-passenger trains) were collected. The 95th percentile value of this stress was 6.75 MPa, which was lower than the compressive stress applied by the strengthening system.

Long-term measurements over a 7-month period

Since the installation of the retrofitting system on the bridge, the long-term performance of the system is being monitored using a WSN system (Decentlab GmbH, Switzerland). The measurements will continue for at least one year. The WSN system consists of two nodes for measuring the strain in each CFRP rod, and one node for the air humidity and temperature measurement, as depicted in Fig. 15. The WSN system collects the sensors' output values, and

sends the data to a platform based in Switzerland every 10 min. The data is accessible for authorised users via the internet.

To measure the strain in each CFRP rod, two strain gauges of type 1-LY66-6/120 (HBM, AG, Darmstadt, Hesse, Germany) were connected to each node using a wired connection: a strain gauge that was applied on the CFRP rod, and a dummy strain gauge to compensate the temperature effect. The strain gauges for the long-term measurements were prepared in advance at Empa, Switzerland, and they were protected against moisture for long-term outdoor application. For detailed information about the WSN system see (Ghafoori et al. 2018).

The prestressing loss in the CFRP rods due to the slippage of the clamping system or CFRP rods inside the wedge-barrel anchors is monitored using the WSN system. The CFRP rods stress evolution during nearly a 7-month period after installation of the strengthening system is shown in Fig. 16(a). The results showed that after an initial drop, the prestressing level in the CFRP rods have stabilised, indicating no strengthening system slippage. The initial drop in the prestressing level is attributed to the fact that the wedge-barrel anchor functions solely through friction, and immediately after the prestressing application, the CFRP rods and wedges move slightly in the barrel. This was also observed in the uniaxial tensile tests on the wedge-barrel anchors. However, after a short time period, the CFRP rod stress level stabilised. Here, a target prestressing level of 995 MPa was selected. The CFRP rods were initially prestressed up to approximately 1050 MPa, and after the initial prestressing loss, the prestressing of approximately 1000 MPa was achieved, showing less than a 5% loss compared to the initial value.

It is noted that the results of the long-term monitoring are valid only for the above-mentioned specific period and not beyond that. During this period, the minimum and maximum air temperature were -3.5 and 36 °C, respectively, and the minimum and maximum relative humidity were 18.7 and 99.2%, respectively. The fluctuations in the CFRP prestressing level in

Fig. 16(a) originate from the air temperature change. As shown in Fig. 16(b), the prestressing change in both CFRP rods are synchronised with the air temperature change. When the temperature increases, the steel elements between the clamping systems expand, and thus, the CFRP prestressing increases. Therefore, in general, an increase in the air temperature leads to an increase in the CFRP prestressing level.

Protection of the strengthening system

Although the CFRP materials have excellent long-term durability in terms of corrosion, fatigue, and show minimum stress relaxation, the mechanical anchors, which were made of steel, need to be protected against corrosion. For protection against corrosion, a layer of Tectyl multi-purpose anti-corrosion coating (Petro-Lubricants-Mineraloel AG, Switzerland) was sprayed on all metallic components of the strengthening system, i.e., the clamping system plates and the wedge-barrel anchors. In addition, in the wedge-barrel anchors, CFRP rods, aluminium wedges, and steel barrels are in direct contact; thus, there is a galvanic corrosion risk. To isolate the wedges, the barrel, and the CFRP rod from the air, a protective cap made of Polyoxymethylene (POM) material (see Fig. 15) was fastened on the free end of the barrel using the threads. The hole in the loading end of the barrel was also filled with a flexible covering agent of type Abdeckkitt AK22 (HBM, AG, Darmstadt, Hesse, Germany) to prevent air and humidity penetration into the anchor. Thus, the wedge-barrel anchors were completely isolated from the surrounding air. Furthermore, to prevent direct rainfall on the clamping system, a galvanised steel cover (i.e., shelter) was installed on top of the clamping system, as shown in Fig. 15(a).

Multiaxial fatigue criterion

The mean stress effect is not clearly considered in the calculation procedure suggested in European and the US standards. Thus, the positive effect of the reduction of the mean stress by the retrofitting systems cannot be considered in such calculations (Heydarinouri et al. 2019). In

addition, the presence of multiaxial stress state in the stringer-to-floor-beam angle connections increases the complexity of the problem. The reasons for overlooking the mean stress effect in codes could be the complexity of the calculations, and also the fact that prestressed retrofitting systems for the riveted structures have been mainly introduced in recent years and are not among the well-established repairing methods (Connor and Lloyd 2017).

In this study, a critical plane-based multiaxial fatigue criterion was proposed to consider the mean stress and multiaxiality at the same time. In this section, first, the multiaxial fatigue theory based on the critical plane approach is briefly discussed. Then, the theory is used to obtain the fatigue state in the angle connections before and after the strengthening.

Critical plane approach

Experimental observations indicate that the cracks nucleate and grow on specific planes called critical planes (Socie and Marquis 2000). Depending on the material and loading conditions, these planes are those with the maximum shear or tensile stress amplitude (Fatemi and Socie 1988). Multiaxial fatigue models that relate fatigue damage to stresses and/or strains on these planes are called critical-plane-based models.

In this study, the modified Wohler curve method (MWCM) proposed in (Susmel and Lazzarin 2002) is employed for multiaxial fatigue analysis. In this model, similar to most stress-based multiaxial models, two parameters are required: the shear stress amplitude and the maximum normal stress on the critical plane. In ductile steels, the driving force for the crack nucleation at the micro level is shear movement. Therefore, the critical plane is the plane with the maximum shear stress amplitude τ_a , as shown in Fig. 17. In addition to the shear stress amplitude, the maximum normal stress acting on the critical plane, $\sigma_{n, \max}$, helps the crack open and propagate at the micro level by reducing the interlocking and friction between the crack surfaces. The MWCM proposes the following equation (Susmel and Lazzarin 2002):

$$\tau_a + (\tau_A - \frac{\sigma_A}{2})\rho = \tau_A \quad (4)$$

where, τ_A is the fully reversed shear fatigue limit, σ_A is fully reversed uniaxial fatigue limit, and ρ is the critical plane stress ratio defined with the following formulation:

$$\rho = \frac{\sigma_{n,max}}{\tau_a} \quad (5)$$

The mean stress effect in MWCM is taken into account with the ρ value. The MWCM diagram is shown in Fig. 18, where τ_α , is plotted as a function of ρ .

As shown in the figure, the τ_α value decreases when ρ increases. However, when ρ is greater than a certain limit, namely ρ_{lim} , a further increase in the ρ value no longer affects the τ_α value. The reason is that when micro cracks are fully open in large ρ values, a further increase in the normal mean stress (accounted for by the ρ value) does not result in an increase in the fatigue damage (Susmel and Lazzarin 2002). The value of ρ_{lim} is mathematically obtained using Eqs. (4) and (5) with the following formulation:

$$\rho_{lim} = \frac{\tau_A}{2\tau_A - \sigma_A} \quad (6)$$

Note that for the case of a fully reversed uniaxial cyclic loading, $\tau_\alpha = \sigma_A/2$ and $\rho = 1$. For design purposes, the value of fully reversed uniaxial fatigue limit is calculated by Johnson's formulation (Johnson 1897; Johnson 1899) as follows:

$$\sigma_A = \frac{S_{ut}}{3} \quad (7)$$

where S_{ut} is the material tensile strength. Moreover, through comparison with the experimental data (Susmel et al. 2005), it has been shown that the fully reversed shear fatigue limit τ_A , where $\rho = 0$, can be estimated by the following equation:

$$\tau_A = \frac{\sigma_A}{\sqrt{3}} \quad (8)$$

Eq. (8) is proposed based on the von Mises hypothesis. To plot the MWCM threshold using Eqs. (6) to (8), only the steel tensile strength is required, which is easily used in design. In Appendix II, the existing experimental data obtained from the uniaxial and multiaxial proportional and non-proportional fatigue tests on mild steel are compared with the proposed design threshold.

Considering Eqs. (6) to (8), independent of the material tensile strength, the value of $\rho_{lim} = 3.73$ is obtained. Assuming $S_{ut} = 400$ MPa for structural steel S235, the MWCM formulation for design can be formulated as follows:

$$\tau_a^* = 77 - 10.3\rho \quad (9)$$

where τ_a^* is the allowable shear stress amplitude on the critical plane for a specific value of ρ . As shown in Fig. 18, the green area below the MWCM diagram is estimated as a safe zone, where a crack is not expected to initiate. Therefore, the point beyond the threshold has to be shifted into the safe zone as a result of the strengthening. The effect of the proposed strengthening system on the stresses is described in the following section.

Multiaxial calculations for the measured cycles

To evaluate the effect of the strengthening system on the fatigue performance of the angle connections in the Aabach Bridge, the MWCM parameters τ_a and ρ , were calculated using the measured stress cycles in the left angle hotspot, as presented in Table 2.

In all of the stress cycles given in Table 2, the stress component τ_{xy} was neglected, and only the stress cycles σ_x and σ_y were considered. In general, the maximum shear stress amplitude was obtained by the calculation of the stress matrix eigenvalues corresponding to the cyclic portion of the stress cycles. As seen in Table 2, the maximum stress amplitude due to different trains are considerably lower than the threshold value of τ_a^* obtained by Eq. (9). Particularly, for the right angle, the ratio of τ_a / τ_a^* is lower than that for the left angle due to the passage of Train (6), showing that the left angle is more critical compared to the right angle.

The stress cycles due to Trains (6) and (23) were considered. The application of 50 kN prestressing force to each of the CFRP rods in the strengthening system reduced the ρ value from 2.36 (due to Train (6)) to 1.45 (due to Train (23)), resulting in approximately a 39% reduction, without a change in the applied shear stress amplitude. Considering the 95th percentile stress values for passenger trains (see Table 1), the ρ value decreased from 2.25 to 1.59 (approximately 30% reduction).

For Train (26), with the stress cycles shown in Fig. 14(c), only the first large cycle was considered. In Table 2, it was assumed that passage of this train over the unstrengthened connections would generate the stress cycles with the same stress range as that after strengthening, without an initial compressive stress due to the prestressing. As presented in Table 2, the strengthening system would reduce the ρ value from 2 to 1.62, (19% reduction). The above-mentioned figures show that the effectiveness of the strengthening system in moving

the points in the MWCM diagram, shown in Fig. 19, depends on the type of train passing over the bridge.

The fatigue state in the left angle due to the passage of the trains given in Table 2, are plotted in the MWCM diagram in Fig. 19. As the strengthening system reduces only the mean stresses, the points primarily follow a horizontal path as a result of the strengthening, i.e., the applied maximum shear stress amplitude remains unchanged and the ρ value decreases.

Determination of the prestressing level in the strengthening system

The prestressing level in the strengthening system was determined in the laboratory tests, as shown in Fig. 3. The set-up was designed so that the fatigue state in the connections are more severe than that in the bridge, as shown in Fig. 19. The stress state in the connection hotspot in the laboratory tests is given in (Heydarinouri et al. 2020). In the laboratory tests, it was observed that through 50 kN prestressing force applied to each of the CFRP rods (100 kN in total), the ρ value decreased from 2.26 to 1.46 (i.e. 35% reduction), with a slight decrease in the τ_α value from 41.6 MPa to 37.5 MPa.

The results of the short-term measurements, shown in Fig. 19, indicate that under a certain prestressing force, the reduction in ρ value for the Train (26), which induced a higher mean stress in the angle, was smaller. Therefore, it is more conservative to consider such trains for design of the strengthening system and determination of the required prestressing force.

Summary and conclusions

In this study, a new retrofitting system for strengthening the stringer-to-floor-beam angle connections in a 92-year old riveted bridge in Switzerland is introduced. In the proposed system, prestressed CFRP rods are used to enable the application in complex details. The mechanical strengthening system consists of a newly developed wedge-barrel anchor to hold the prestressed

CFRP rod and a clamping system attached to the stringer flange, transmitting the loads solely through friction. The key conclusions of the current study are as follows:

- The strengthening system was installed using the space under the bridge floor system, with no interruption of traffic on the bridge. For the on-site presetting of the wedges, a new system was introduced, where there was no need for the hydraulic jack to apply the presetting force.
- For prestressing, two 8 mm-CFRP rods were simultaneously pulled using a hydraulic pump, and a temporary housing. The prestressing force at each CFRP rod was 50 kN (equivalent to 995 MPa), which is 49% of the guaranteed tensile strength of the rods. After installation, all parts of the strengthening system were protected against corrosion using the anti-corrosion coatings and isolation from the surrounding environment.
- Short-term measurements indicated that the strengthening system reduced only the mean stress, without changing the stress range magnitude.
- For at least one-year after installation, the long-term performance of the strengthening system is being monitored using the WSN system. The measurements show that during a 7-month measurement period since installation, no prestress loss has been observed.
- A critical-plane-based multiaxial fatigue criterion (i.e., the MWCM) was used to evaluate the fatigue state in the connections before and after strengthening. The MWCM threshold can be plotted using only the material tensile strength, making it easy to use in design. The results of the short-term measurements show that the applied stresses due to the passage of different trains are smaller than 30% of the fatigue threshold, i.e., $\tau_{\alpha} / \tau_{\alpha}^{*} < 0.3$.
- The mean stress reduction by the strengthening system is considered using the ρ value in the MWCM, which depends on the type of trains; for 95th percentile stress values for the passenger trains, the ρ value decreased from 2.25 to 1.59 (approximately 30% reduction),

while for Train (26), as a non-passenger train, the ρ value was reduced from 2 to 1.62 (19% reduction).

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Appendix I. Stress history in the web for Trains (6) and (14)

The stress history due to the passage of Trains (6) and (14) in the stringer web is depicted in Fig. 20.

Appendix II. Multiaxial fatigue tests

In this section, the design threshold according to the MWCM criterion is compared with the experimental results from the fatigue tests on mild steel, under uniaxial and multiaxial loading conditions. A series of multiaxial tests on mild steel performed by Nishihara and Kawamoto 1945 were reanalysed by McDiarmid 1987 using the critical plane theory. These data were later used by Susmel et al. 2005 to check the accuracy of the proposed MWCM diagram to predict

the fatigue failure, as shown in Fig. 21(a). In the experimental results shown in this figure, the specimens were subjected to the in-phase and out-of-phase fully reversed σ_x and τ_{xy} stresses.

In general, to be plotted, the MWCM diagram needs to be calibrated with two series of experimental data. For the experimental data shown in Fig. 21(a), $\sigma_A = 235.4$ MPa and $\tau_A = 137.3$ MPa. Therefore, the MWCM diagram can be plotted using Eqs. (4) and (6), considering the fact that the tensile strength of the steel used in the tests was $S_{ut} = 382$ MPa. As shown in the figure, the experimental data are well above the design threshold.

The MWCM diagram is also compared with the experimental results under uniaxial cyclic stresses with non-zero mean stress taken from Frost et al. 1974, as given in Fig. 21(b). The MWCM threshold has been calibrated with two experimental datasets, i.e., one with zero mean stress and the other with non-zero mean stress. Using the tensile strength value of $S_{ut} = 410$ MPa for the steel used in the tests, the design threshold illustrated in Fig. 21(b) shows the state of the design threshold compared to the experimental data.

Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request. The items are listed as follows: the data of the short- and long-term measurements, as well as the MATLAB code used for calculation of the stress history.

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Table 1

Table. 1. Cyclic stresses in the left angle hotspot due to the passenger trains

	$\Delta\sigma_x$ (MPa)	σ_{mx} (MPa)	$\Delta\sigma_y$ (MPa)	σ_{my} (MPa)
Average	29	17	10.6	6.6
SD	4.8	3	2.3	2
S95%	36.7	22.9	14	9.8

Table 2

Table 2. MWCM parameters before and after strengthening in the left angle hotspot

Strengthening	Angle	Train	Stress (MPa)				MWCM parameters			
			$\Delta\sigma_x$	σ_{mx}	$\Delta\sigma_y$	σ_{my}	ρ	τ_α (MPa)	τ_α^* (MPa)	$\tau_\alpha / \tau_\alpha^*$ (%)
Without	Left	(6)	36.2	26.4	10.4	6.8	2.36	9.05	52.6	17.2
	Right	(6)	5	22	-5	5	2.59	4.25	50.3	8.4
	Left	(26)*	63.7	31.9	18	12.5	2	16.25	50.7	28.8
	Left	$S_{0.95}$	36.7	22.9	14	9.8	2.25	9.2	53.8	17
With	Left	(23)	36.5	8.25	12	-8.5	1.45	9.13	62	14.7
	Left	(26)	63.7	19.9	18	-3.5	1.62	15.9	60.3	26.4
	Left	$S_{0.95}^*$	36.7	10.9	14	-6.2	1.59	9.2	60.6	15.1

*The values were obtained not from the measurements but based on the assumption that the strengthening system would reduce only the mean stress by 12 and 16 MPa in the x- and y- directions, respectively.

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

(a)

(b)

(c)

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11

Figure 12

Figure 13

Figure 14

Figure 15

(a)

(b)

Figure 16

Figure 17

Figure 18

Figure 19

Figure 20

Figure 21

Figure captions list:

Fig. 1. (a) Global deflection of the bridge; Secondary tensile stress due to semi-rigidity of the connections with the tensile stress in the: (b) top of the connections; (c) bottom of the connections; (d) Secondary tensile forces in the connections due to the interaction between stringers and bottom chords; (e) Fatigue cracks observed in the double-angle web-to-web connections (Haghani, Al-Emrani et al. 2012).

Fig. 2. Stringer-to-floor-beam double angle connection: (a) Geometric complexity; (b) Different components of the strengthening system.

Fig. 3. Set-up for the laboratory tests.

Fig. 4. Aabach Railway Bridge, Lachen, Switzerland.

Fig. 5. Details of Aabach Bridge: (a) Layout of the bridge deck; (b) Dimensions of the bridge frame; (c) Elements of the bridge (bottom view).

Fig. 6. Presetting procedure: (a) Passing the CFRP rod through the hollow bolt; (b) Installing the clamping system before presetting; (c) Pushing the wedges into the barrel.

Fig. 7. Prestressing procedure: Pumping the hydraulic cylinders to pull the CFRP rods using a temporary housing.

Fig. 8. Applied prestressing to the CFRP rods during the oil pumping.

Fig. 9. Strain gauges used for short-term measurements: (a) Rosette strain gauges on the angles; (b) Strain gauges on the web and the CFRP rod.

Fig. 10. Directions of the rosette strain gauge on the left angle: (a) Top view of the angle and coordinate system; (b) Corresponding Mohr's circle.

Fig. 11. Strain history in the hotspot of the left angle before strengthening: (a) Due to the passage of different trains; (b) Due to the passage of Train (6).

Fig. 12. Stress history in the hotspot of the left angle: (a) Before strengthening; (b) After strengthening.

Fig. 13. Stress history due to (6) and (23) in left angle: (a) x-direction; (b) y-direction; (c) Shear stress.

Fig. 14. Stress history in the left angle due to: (a) Train (14); (b) Train (19); (c) Train (26).

Fig. 15. WSN system for the long-term monitoring and protective measures against corrosion: (a) Left side; (b) Right side.

Fig. 16. Long-term measurements over a 7-month period after the installation: (a) Stress history in the CFRP rods measured by the WSN system; (b) Stress variation in the CFRP rods with the air temperature change.

Fig. 17. Effective parameters in fatigue crack nucleation in the MWCM.

Fig. 18. Fig. 18. Modified Wöhler curve method diagram.

Fig. 19. MWCM diagram to evaluate the fatigue state before and after strengthening in the left angle.

Fig. 20. Stress history in the web of the stringer: (a) Train (6); (b) Train (14).

Fig. 21. Comparison of the MWCM diagram with the experimental results generated: (a) from plane specimens under multiaxial fully reversed in-phase and out-of-phase bending and torsion; (b) under uniaxial loading with zero and non-zero mean stresses.