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## **Fatigue Strengthening of Steel Bridges with Adhesively Bonded CFRP Laminates: Case Study**

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# 1 **Fatigue strengthening of steel bridges with adhesively bonded CFRP**

## 2 **laminates: case study**

3  
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31

## 32 **ABSTRACT**

33

34 One of the goals of applying sustainable development to bridge infrastructure is to provide bridge  
35 owners with strengthening solutions that may lead to increased service life for existing structures. In the  
36 case of steel bridges, assessment of the remaining service life is most often linked to the determination  
37 of structural deterioration caused by corrosion and fatigue. Damage caused by fatigue is very difficult  
38 to assess before crack initiation, and is more likely to occur in old structures, for which the phenomenon  
39 was not taken into account in designs before 1970. In addition, old steel materials display more brittle  
40 behaviour. In answer to these challenges, a preventive methodology for fatigue strengthening of steel  
41 structures was developed. The method begins with scheduling a fatigue design analysis of the existing  
42 construction to determine the most fatigue damage exposed construction elements of the bridge. The  
43 remaining fatigue life of these elements can be increased with a strengthening solution based on the use  
44 of adhesively bonded ultra high modulus (UHM) carbon fibre-reinforced polymer (CFRP) plates, which  
45 is applied to a steel surface before failure indicators such as cracks arise. This article presents the  
46 development process of this preventive method and a demonstrative application to an existing bridge  
47 (Jarama Bridge). Strain measurement was carried out to verify the theoretical expectations of the  
48 reinforcement. Different parameters were studied, including the influence of low traffic volumes during  
49 reinforcement application. The results proved the efficiency of this system for the structure under study.

50

51 **Key words:** Strengthening, fatigue, steel bridges, CFRP composites, on-site application, life extension.

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53

54

## 55 **Introduction**

56

57 There is a strong need worldwide to develop and assess sustainable solutions to increase the life  
58 expectancy of existing infrastructure. This is particularly true in the case of steel bridges both in Europe  
59 and in the USA (Lee 2012) (Ye et al. 2014). Most of these structures were constructed according to old  
60 standards in which fatigue was not considered, despite fatigue being the second main pathology after  
61 corrosion affecting steel structures (Palmer 2014). Most of the bridges of concern are approaching the  
62 end of their designed service life. In addition, they have often been subjected to traffic substantially  
63 above the amount anticipated most likely significantly decreasing their life expectancy (Kühn et al.  
64 2008). To date, the mainstream approach against fatigue problems was centred on a reactive strategy  
65 based on maintenance or repair operations undertaken after the occurrence of cracks in the structure.  
66 With an aging bridge stock, it is necessary to change this paradigm and widely adopt a preventive  
67 strategy. Ultimately this approach will support a more sustainable management of life expectancy of  
68 steel structures (Ghafoori 2019) (Orcesi et al. 2019).

69

70 Existing reinforcement or repair methods for steel structures are mostly based on the installation of  
71 additional steel plates attached either by riveting, bolting or welding (FHWA 2013). These methods  
72 have the disadvantage of adding a large additional weight, are difficult to implement (labour-intensive  
73 and disruptive to traffic), and may decrease fatigue life expectancy due to local stress concentrations in  
74 the connection areas (Karbhari 2014). Carbon Fibre-Reinforced Polymer (CFRP) composites, though  
75 more expensive than steel plates, present several relevant advantages making them suitable and cost  
76 effective for steel bridge retrofitting. Research has already proven the efficiency of CFRP composites  
77 particularly with regards to fatigue (Dawood et al. 2007) (Kim et al. 2011) (Kamruzzaman 2014). CFRP  
78 installation is less time consuming as compared to traditional repair solutions, typically a few days to a  
79 month, therefore causing fewer traffic disruptions (Peiris 2015). Such composite materials additionally  
80 exhibit a high strength-to-weight ratio, excellent fatigue properties, and high durability and versatility,  
81 and are easy to handle and apply without the need of heavy equipment (Miller et al. 2001)  
82 (Tavakkolizadeh et al. 2003) (Zhao et al. 2007) (Bocciarelli et al. 2008). Their attachment to the existing

83 structure can be achieved either by adhesive bonding or by using frictional equipment (Kianmofrad et  
84 al. 2017). A number of guidelines have been recommended to ensure improved quality of CFRP  
85 installation on steel structures (Moy 2001) (Cadei et al. 2004) (Schnerch et al. 2007) (CNR 2007) (DNV  
86 2012). Previous investigations identified two main options for the design of cost-effective CFRP  
87 strengthening systems to be applied to steel structures (Ghafoori et al. 2015a): Ultra High Modulus  
88 (UHM) CFRP reinforcement (Schnerch et al. 2007) or prestressed CFRP composite plates (Ghafoori et  
89 al. 2015b) (Ghafoori et al. 2016). Since the first documented application of adhesively bonded CFRP  
90 reinforcements for the strengthening of the Tickford Bridge in the UK (Lane et al. 2000), only a small  
91 number of on-site demonstration projects have been reported in literature (Miller et al. 2001) (Luke  
92 2001) (Hollaway et al. 2002) (Moy et al. 2007) (Zhao 2013) (Moy 2014) (Peiris et al. 2015) (Ghafoori  
93 et al. 2015c) (Ghafoori et al. 2018). Disappointingly, the actual use of application of adhesively bonded  
94 CFRP reinforcements to civil steel structures remains limited.

95

96 Recently, a preventive strategy regarding fatigue of steel structures, relying on the use of adhesively  
97 bonded CFRP reinforcement, was developed (Wahbeh et al. 2018 a). This strategy relies mostly on a  
98 precise diagnosis of the structure, a remaining fatigue assessment methodology, and a specific  
99 adhesively bonded reinforcement process based on the use of a commercially available UHM CFRP  
100 plate combined with a novel formulated polymer adhesive intended to provide enhanced adhesion on  
101 steel substrates and improved resistance to fatigue, humidity aging and high temperatures (Chataigner  
102 et al. 2018). Of course, due to the fixed shape and the high stiffness of the UHM CFRP plate, such a  
103 reinforcement solution is only adapted to plane surfaces and is not compliant with non-planar critical  
104 details (Hu et al. 2017). The whole methodology was applied on an actual steel bridge, the Jarama  
105 Bridge, which is presented herein.

106

107 The first section of this article aims to describe the steel bridge on which the methodology was applied,  
108 in order to demonstrate and assess the process. The second portion is devoted to the application of the  
109 developed methodology to this structure. This includes its preliminary diagnosis, the initial residual life  
110 assessment, a short description of the reinforcement system, and the design of the reinforcement. The

111 third section addresses the field application of the solution and the monitoring used to assess its  
112 efficiency. In the last section, the realized load tests and their results are presented and discussed.

113

## 114 **Presentation of the studied steel bridge**

115

### 116 *General overview*

117

118 The Jarama Bridge is a five-span steel bridge carrying two lanes of traffic on Road M-111 over the  
119 Jarama River in Madrid, Spain, connecting the Barajas-Madrid Airport to Paracuellos de Jarama. The  
120 bridge has a central three-span continuous segment and two simple side-span segments (figures 1 and  
121 2). The bridge superstructure consists of two main longitudinal built-up I-section steel plate girders with  
122 transverse cross-bracing and floor beams. The two main girders were partially pre-fabricated and finally  
123 assembled and welded in the field. The connections between the main girders and additional steel  
124 elements (e.g. cross-bracing, floor beams) were assembled and welded on-site. The deck of the bridge  
125 consists of a non-composite concrete slab simply supported by the steel girders. The girders are  
126 supported by neoprene bearings on the abutments, and fixed and movable steel plate bearings on the  
127 piers.

128

129 The bridge was designed in 1962 following the 1956 Spanish Design Code, and was built around 1965  
130 to replace the old bridge over the Jarama River. Existing bridge plans and documentation show that the  
131 main structural elements were manufactured using F-622 steel (UNE 1981). The material properties of  
132 the F-622 steel were similar to the A-42B steel and comparable to the actual S-275 steel with an elastic  
133 limit of 255 MPa and rupture strength between 410-520 MPa. The top and bottom flanges of the main  
134 I-shaped girders in the central continuous spans have a width of 700 mm and a thickness of 30 mm. The  
135 girder web thickness is 15 mm, while the height varies between 1,870 mm and 2,810 mm. The girders  
136 in the simply supported spans have 200×45 mm flanges and 1,870×12 mm webs.

137

138 The steel elements of the Jarama Bridge were generally in a good condition. However there was  
139 evidence of local deficiencies including moisture, moss, corrosion. On the top flange of the main girders,  
140 general corrosion was noted. In some cases, the corrosion had already progressed into the web plates.  
141 In other cases, efflorescence and peeling problems were discovered. The same deficiencies were also  
142 present on the bottom flange. Crosses or x-bracing appeared to be in good condition and had no visual  
143 defects. Moreover the bottom lateral bracing members of span 1 showed obvious distortion, corrosion  
144 and other defects probably caused by the impact of objects. One angle in the bottom lateral bracing  
145 member of span 5 also presented a small deformation.

146

147 The floor beams were in good condition. Moisture, efflorescence and corrosion on the top flange were  
148 noted locally. Some areas of damaged and peeled coating could be seen in the bottom flange. Larger  
149 areas affected by moisture and corrosion were located in the floor beams over piers and abutments.

150

#### 151 *Specific issues regarding material characterization and fatigue*

152

153 The bridge design did not incorporate any fatigue criteria since there were no fatigue requirements in  
154 the 1956 code. However, the structural steel elements were designed with a tension limit of  $137 \text{ N/mm}^2$   
155 which correspond approximately to a 1.80 safety factor—the most conservative safety factor according  
156 to the 1956 design code—to cover all effects.

157

158 The prefabricated sections of the main steel girders were initially welded at the workshop and then  
159 transported to the job site, where they were assembled and welded in place. Overall, a total of 36 butt-  
160 welded joints were welded in the factory, and another eight field butt-welded joints were welded on-site  
161 on both main girders. The resulting stress concentrations in the butt-welded splices can produce a  
162 reduction in the fatigue capacity of the girders. The presence of these fatigue-prone details in the main  
163 girders makes the Jarama Bridge a suitable candidate structure for implementation of a preventive  
164 fatigue strengthening of steel structures with adhesively bonded CFRPs. As part of the proposed retrofit,  
165 a detailed bridge inspection was performed on the Jarama Bridge in 2017; visual inspections and

166 nondestructive tests (i.e. ultrasonic tests) were performed on four on-site butt-welded splices and 27  
167 splices welded in the factory. Results from the visual inspections and ultrasonic tests showed that the  
168 selected on-site welds did not comply with the quality standards according to AL2 (UNE 2011) or level  
169 B (UNE 2014). Level B means that there are high-level imperfections, and level AL2 is an ultrasonic  
170 acceptance level for full penetration welded joints in ferritic steels corresponding to level B quality  
171 according to CEN (2014). Eleven of the in-shop welds also failed to meet the quality standards under  
172 AL2. These observations, summarized in Table 1, conclude definitively that the on-site and in-shop  
173 butt-welded splices do not meet current quality standards. This is not surprising given the advancement  
174 in bridge welding codes and improved implementation over the past 50 years.

175

176 Eight steel test coupons taken from girders have been tested for material characterization, including  
177 mechanical, chemical and metallographic examination. Chemical and metallographic results show that  
178 none of the test coupons meet the standards of F-622 classification with accordance to CSIC (1969), nor  
179 A-42b characteristics with standards (COAM 1964). During evaluations in accordance with UNE  
180 (2006), six tested coupons met the quality standards and only two did not. A tensile strength test, Brinell  
181 hardness test and Charpy impact test were also performed. Four test coupons were obtained from the  
182 transverse stiffener of girders. All of them met F-622 characteristics in accordance with CSIC (1969),  
183 A-42 b in accordance with standards from COAM (1964), S235JR and S275JR in accordance to UNE  
184 (2006). Four more coupons were cut out from the bottom flange of the girders. One of them met the  
185 characteristics of F-622 and A-42ba. Two of them meet S235JR quality in accordance with UNE (2006).  
186 The remainder of the test coupons did not meet any quality requirements.

187

## 188 **Description of the proposed methodology to prevent fatigue damage**

189

### 190 *Preliminary diagnosis of the structure*

191

192 The most critical locations regarding fatigue of steel structures are the assembly joints (Kühn 2008)  
193 (FHWA 2013) (Palmer 2014). As the presented application is part of a demonstration, only eight shop  
194 butt-welds on both main steel girders in one of the central spans (span 2) were considered adequate (no  
195 defects, easy access) and initially selected to be strengthened using the developed adhesively bonded  
196 CRFP reinforcement. However, because of the proximity of two of these pre-selected welds to the kink  
197 of the bottom flange, it was decided to exclude them from the investigation. Figure 3 shows the location  
198 of the six shop butt-welds (welds 29, 26, 25, 20, 17, and 16) that were strengthened and evaluated in  
199 this investigation. An additional denomination of those six locations is also used in this document in  
200 order to remind the span (S), beam (B) and weld (W) locations. Thus, the weld n°29 is also denominated  
201 S2-B1-W1 as it is the first butt weld located on the first beam of the second span. The correspondence  
202 between weld number and this identification is reminded for each of the six studied welds in Figure 3.

203

204 Obviously, in the case of an actual bridge requiring full evaluation, the entire structure should be  
205 analyzed and the assessment of residual service life determined for each element and joint. This would  
206 allow identification of the most critical locations in need of reinforcement.

207

### 208 *Assessment of residual fatigue life*

209

210 The fatigue resistance and remaining fatigue life of the remaining girders were estimated using the  
211 FASSTbridge fatigue assessment methodology that had been developed to provide a reliable and  
212 preventive fatigue assessment of existing steel bridges in a pre-failure condition, and calculate the life-  
213 time expectancy (Wahbeh et al. 2018b) (Figure 4). The FASSTbridge fatigue assessment tool was  
214 developed based on both AASHTO (AASHTO 2012) and Eurocode (CEN 2002) specifications and  
215 consist of a questionnaire purposely created to gather both quantitative and qualitative information about  
216 the bridge and the specific detail being analyzed. In this approach, the questionnaire inputs have been  
217 limited to pre-determined values in order to generate a set of modification factors ( $\alpha$  values) for different  
218 fatigue design variables, and to perform the required fatigue assessment calculations. Table 2 and Table

219 3 summarize the fatigue design variables based on AASHTO and Eurocode specifications and  
220 corresponding proposed modification factors ( $\alpha$  values).

221

222 The estimation of fatigue capacity and remaining fatigue lifetime were performed according to the  
223 Eurocode recommendations. The results for the six studied locations are shown in Table 4. These results  
224 highlight that only one of the studied locations seems to be critical regarding fatigue (which is consistent  
225 with the real status of the bridge: no fatigue damage detected). Yet, in order to study the efficiency of  
226 the reinforcement and to investigate different application parameters, it was decided to apply the  
227 developed reinforcement on all six locations.

228

### 229 *Description of the strengthening system*

230

231 The developed strengthening system relies on the use of a commercial UHM CFRP plate provided by  
232 Epsilon Composite (France). A specific bi-component hybrid epoxy/polyurethane adhesive was  
233 developed for this application by Collanti Concorde (Italy). In order to obtain the required specification  
234 of glass transition temperature  $T_g > 71^\circ\text{C}$ , which is at least  $15^\circ\text{C}$  above the maximum service temperature  
235 expected on steel bridges in Europe (CEN 2002), this adhesive needs to be post-cured after application  
236 at  $80^\circ\text{C}$  for one hour. An extensive experimental program enabled the whole system behavior to be  
237 checked, including determining its design characteristics, its sensitivity and replicability, its dependency  
238 to temperature, and its durability both regarding moisture and fatigue. More details on these issues can  
239 be found in Chataigner et al. (2018).

240

### 241 *Design of the reinforcement*

242

243 Three different CRFP strengthening configurations with a varying numbers of CFRP plates, numbers of  
244 layers, and lengths were evaluated. A maximum of five CFRP plates per weld were selected based on  
245 the width of the girder bottom flange and a 30 mm minimum separation between consecutive CFRP  
246 plates. A minimum length of 1,200 mm was adopted for the CFRP plates. This allows for covering of

247 the butt weld to reinforce (200 mm) including twice the required anchorage length of 120 mm  
248 determined in Chataigner et al. (2018), and an additional safety length in order to cope with the creep  
249 of anchorage length with aging (380 mm on each side).

250

251 A preliminary investigation of the effects of the geometry of CFRP plates indicated the influence of the  
252 sheet length is negligible provided it is superior to the characteristic anchorage length (Nozaka et al.  
253 2005). Using a CFRP plate with a length of 1,200 mm led to similar results as employing a CFRP plate  
254 over a longer span for the considered application of this study (localized butt weld reinforcements). In  
255 this preliminary analysis, it was assumed only normal longitudinal stresses were present in the section  
256 and the CFRP and steel would act compositely as a one unit.

257

258 For the strengthening configurations with several layers of CFRP plates, an additional anchorage length  
259 of 100 mm was added to each end of the CFRP panel. Details of all six strengthening configurations  
260 evaluated in this investigation are summarized in Table 5 and Figure 5. Each of the CFRP plates  
261 employed in all strengthening configurations had a thickness of 4.0 mm, a width of 100 mm, and a  
262 modulus of elasticity of 460,000 N/mm<sup>2</sup>. The investigations in Chataigner et al. (2018) were used to  
263 extrapolate the ultimate acceptable stresses within the applied reinforcement (1,074 microstrain for one-  
264 layer configuration, 537 microstrains for two-layer configuration, and 358 microstrains for three-layer  
265 configuration). Taking into account the fatigue resistance of the adhesively bonded assembly, it was  
266 necessary to adopt 30% of this ultimate value in service to ensure a fatigue level under the fatigue  
267 endurance limit.

268

269 To evaluate the performance of each strengthening configuration with CFRP plates on butt welds, a  
270 series of finite element analyses (FEAs) were conducted in order to obtain the differences between the  
271 minimum and maximum stresses in the six butt welds due to fatigue loading for the reference  
272 (unstrengthened) and strengthened structural configurations (Diez et al. 2017). As expected, there was  
273 a correlation between the reduction in the tensile stresses in the welds and the number of layers and

274 CFRP plates. As the number of layers and the number of CFRP plates installed increased, larger  
275 reductions on the stress ranges were observed.

276

277 For the strengthening configuration of four CFRP plates and two layers per weld, a reduction of  
278 approximately 8-15% in the stresses were achieved. When five CFRP sheets were used in a single layer  
279 configuration, a 4-7% reduction in the stresses in the weld were estimated. The largest reduction of  
280 approximately 21% was achieved with a strengthening configuration using three CFRP plates in three  
281 layers.

282

283 Based on these results, the remaining fatigue life has been determined for the six studied locations for  
284 the strengthened situation using a finite element to obtain stress reduction and the developed remaining  
285 fatigue life tool (Wahbeh et al. 2018 b). This fatigue assessment tool was developed according to  
286 NCHRP (2012) through a simple questionnaire for bridge owners which takes into account several  
287 parameters on fatigue class details such as state of damage or real traffic loads. The results from the  
288 studied details are presented in Table 6. For the only critical location (Weld N° 17), the remaining fatigue  
289 life seems to be almost tripled due to the reinforcement, indicating the efficiency of the proposed  
290 solution.

291

## 292 **Application of the strengthening solution in the field**

293

### 294 *Installation of CFRP systems*

295

296 The installation of the bonded reinforcement was carried out according to the recommendations of  
297 Schnerch et al. (2007) and Italian National Research Council (CNR 2007). Prior to installation of the  
298 CFRP plates, the steel surface was initially grit blasted to remove coatings (painting), corrosion  
299 products, and any other particles that could affect the bond properties between the CFRP adhesive and  
300 the steel. In the case of lead paint, as is often encountered in old steel structures, this process may require  
301 extensive protective measures during surface preparation operations. In the case of Jarama Bridge, this

302 was not the case, and classical methodologies could therefore be adopted. The surface preparation was  
303 performed following CEN (2007), to achieve a level of Sa 2 1/2 required for the system. The welds were  
304 also grinded to eliminate any geometric disruptions in the weld and obtain a flat surface. After grit  
305 blasting and weld grinding, the entire steel surface was cleaned and degreased.

306

307 The CFRP plates are up to 1,600 mm long, 4-mm thick, and up to three layers, as previously reported  
308 in Table 5. The material is UHM CFRP ( $E = 460$  GPa). Because the stiffness of the CFRP plates is  
309 relatively high, and even more so when applied in several layers, the adaptability of the plates to the  
310 geometry of the steel plates is very low, and therefore any geometric deviations from a plane (both  
311 longitudinal and transverse) or other irregularities must be accommodated by the adhesive. The  
312 thickness of the adhesive was measured and found to be at least one mm; analysis and tests indicated no  
313 reduction of capacity for thicknesses up to 3 mm (Chataigner et al. 2018). For each CFRP plate, a  $45^\circ$   
314 tapering was carried out on each edge as recommended in Schnerch et al. (2007).

315

316 In the case of the Jarama Bridge, it was decided to retrofit the bottom flange of the girders which are in  
317 tension and therefore susceptible to fatigue cycles. Since the CFRP plates are to be installed underneath  
318 the girders, it was necessary to use a fixation system that would be able to keep the plates in place during  
319 the curing and hardening of the adhesive. First, the adhesive mix was applied directly to the surface of  
320 the CFRP plate after removal of the peel-ply. To spread the adhesive uniformly, a device similar to a  
321 trowel with a v-notch was used (Figure 6a). In the case of the multi-layer configurations, the  
322 prefabrication option was selected. Previously, at the workshop or on-site, the layers were attached to  
323 each other in the predefined arrangement, and pressure was then applied with small rollers to ensure all  
324 air bubbles or gaps were removed (Figure 6b). The CFRP plate with the adhesive on one side must be  
325 pressed into the steel starting from one end and gradually moving towards the other to allow air to  
326 escape. A laminate roller was used from the center of the plate towards the ends to remove as much air  
327 as possible and ensure good contact between the CFRP, adhesive and steel substrate. The excess resin  
328 that escaped during pressing was removed, leaving a  $45^\circ$  fillet as recommended in Schnerch et al. (2007).  
329 Care was taken to ensure that not too much adhesive would be pressed out from the joint at the plate

330 ends, and that the adhesive thickness in the end regions was consistent with the rest of the plate. Once  
331 the CFRP, adhesive and steel surface were completely in contact, the plates were clamped to the steel  
332 structure with the fixation system developed *ad hoc*.

333

334 As described in Chataigner et al. (2018), a post-curing of the system is needed (one hour at 80 °C) in  
335 order for the adhesive's glass transition temperature to reach the required value of 71 °C without losing  
336 properties. Ceramic pads were used to transfer the heat to the system to reach the target curing  
337 temperature of 80 °C and maintain it constant during the curing time (one hour). The ceramic pads were  
338 placed to completely cover the surface of the CFRP plates (Figure 7). Thermocouples were installed on  
339 the steel surface close to the CFRP plates and were used to control the curing temperature. Isolation  
340 blankets were also used to reduce heat loss, maintain the required temperature inside the system, and  
341 improve energy efficiency. The post-curing process started a minimum of two hours after installation of  
342 the CFRP plates.

343

344 After removal of the heating system, paint was applied to the whole reinforcement and surrounding steel  
345 to protect the area where the coatings had been removed. The selected coating had been previously  
346 assessed through experimental investigations to ensure its compatibility with both the steel surface and  
347 CFRP plates.

348

349 Each of the six weld locations were reinforced during nighttime to reduce traffic disruptions. The  
350 duration of the application (including post-curing) lasted between 4.5 to 6.5 hours, depending on the  
351 precise weld location and CFRP configuration, and the influence of the learning curve for the  
352 installation. For three locations (welds n°17, 20 and 25), traffic was partially deviated during the entire  
353 application (only one traffic lane open instead of two lanes). For two locations (welds n°26 and 29),  
354 traffic was deviated only during the application of the bonded CFRP plates before post-curing. For one  
355 of the locations (weld n° 16), traffic was not deviated at all.

356

357 *Description of the monitoring instrumentation*

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Before applying the reinforcement, electro-mechanic strain gauges were installed at the six studied locations. For each location, four strain gauges were installed on top of the bottom flange in the longitudinal direction. A first pair of strain gauges was installed 80 mm from the studied butt weld. One of the gauges was bonded to the flange, while the second was bonded to a separate piece of metal to compensate for temperature effects due to a half-bridge circuit (Figure 8). The second pair was installed 950 mm from the studied butt weld, outside of the reinforced zone. The denomination of the strain gauges in the rest of the article will be X-1 and X-2, corresponding respectively to the first axial strain at 80 mm from the studied weld and the second axial strain at 950 mm from the studied weld, and X corresponding the weld number. The strain gauges received a silicone protective cover to shield them from adventitious mechanical interaction (i.e. impact, bump) during the strengthening intervention. An HBM Spider8 data acquisition system with a 200 Hz measurement rate was used, which allowed the simultaneous recording of all 12 half-bridge circuits. All strain data were recorded with reference to the bridge state without traffic loads (“differential strain”). “Positive” strains correspond to additional tension, while “negative” strains correspond to reduced tension in the girders with respect to the initial condition.

A recently developed debonding sensor was also installed during the application of the CFRP reinforcement. It is described in detail in Lehmann et al. (2019). This debonding sensor was installed during the strengthening intervention in order to verify its applicability. It consists of an ultra-thin square element that needs to be inserted into the bond line close to the edge (Figure 9). Preliminary laboratory experimental investigations confirmed the insertion of this sensor did not induce premature debonding (Lehmann et al. 2019). No difficulty was encountered during the application onsite, and no debonding was detected during any of the led investigations. The sensors will need to be investigated at a later date to verify whether debonding occurred during the structure’s service life.

**Efficiency assessment of the reinforcement**

386 *Description of the bridge loading tests*

387

388 A fixed load test was performed in order to gain knowledge about the condition of the Jarama Bridge.  
389 By measuring strains both before and after the bridge retrofit, this fixed load condition test was able to  
390 effectively demonstrate the effectiveness of the intervention. The tests took place during the two nights  
391 immediately before and after the retrofitting. All of the load tests were repeated for both lanes on the  
392 bridge even when one lane had to be blocked.

393

394 First, a static fixed load test was performed including three different truck positions. The static test was  
395 performed by placing two 27-ton trucks (Renault Kerax) in three predefined positions (Figure 10),  
396 carrying out the strain measurements after stabilizing the load. The initial position was chosen to obtain  
397 maximum strains in welds 17, 20, 26 and 29. The second position was chosen to obtain maximum  
398 positive strains in welds 16, 17, 25 and 26. The third position was chosen to obtain maximum negative  
399 strains in the studied welds.

400

401 Two dynamic load tests were then performed by placing a wooden plank, four cm in height, in two  
402 selected positions, over which a single truck passed at speeds of 20 km/h and 50 km/h (Figure 11). The  
403 passage of the truck when hitting the plank induced a vertical dynamic impact load in the structure. No  
404 take-off and break loading scenarios were carried out in the presented case, although these may also  
405 have a strong impact on the measured stresses as detailed in (Hosseini et al. 2019), due to its potential  
406 to increase the static stresses measurements from 50%.

407

408 *Obtained results and discussion*

409

410 An example of strain recording during the positioning of the two trucks on the bridge for a static load  
411 test is provided in Figure 12 (the first position load case is given in figure 10). As anticipated, the first  
412 studied position, welds 20, 29, 26 and 17 are subject to the highest strain. Since only one side of the  
413 bridge was blocked to traffic, there are some small perturbations of the measurement due to the traffic

414 on the other side. However, as the plateau value is considered, these perturbations do not affect the  
415 obtained results. For each weld location, the maximum measured differential strains before and after  
416 reinforcement are provided in tables 7 and 8.

417

418 An example of strain recorded during the dynamic load tests is provided in Figure 13. A clear dynamic  
419 effect may be observed as the strain levels reached are almost as high as those obtained in static tests,  
420 though the applied load is half as high. Furthermore, the speed of the truck had a slight effect on the  
421 results, resulting in an increase in the dynamic effect. For each weld location, the maximum measured  
422 differential strains before and after reinforcement obtained during dynamic tests are also provided in  
423 tables 7 and 8.

424

425 For each of the six locations, strains were reduced thanks to the CFRP reinforcement both close to the  
426 weld toe (Table 7) and far from the weld toe (Table 8). This experimentally observed reduction seems  
427 to be higher than the theoretically anticipated reduction through FEA. This was attributed to a local  
428 bending effect due to the asymmetric reinforcement geometry. This also explains the observed strain  
429 reduction on the second location (X-2) situated far from the weld toe, although it was situated outside  
430 the reinforced area. Static and dynamic results appeared to be very similar except for weld 26.

431

432 As the most critical fatigue zone is the location close to the weld (X-1), corresponding strain  
433 measurements are represented in Figure 14. The highest strain reductions (20 to 30 %) were obtained  
434 for locations reinforced with the highest number of CFRP plates, and the proposed solution worked well  
435 in the case of the three-layer configuration (welds 16 and 25). In addition, there was no discrepancy in  
436 the locations reinforced without total traffic disruption (welds 16, 26 and 29). This suggests that the  
437 proposed solution could be applied under low traffic, which would be a critical advantage of this  
438 strengthening method.

439

440 **Conclusions**

441

442 As far as the fatigue of steel structures is concerned, most of the time, structure owners employ a curative  
443 strategy after the detection of cracks in the most critical details. The presented application is aimed at  
444 highlighting the possibility of utilizing a preventive strategy before fatigue crack initiation to increase  
445 the lifetime of steel structures. From this demonstration on the Jarama Bridge in Madrid, Spain, several  
446 conclusions were obtained:

- 447 • The developed fatigue assessment methodology allowed for identification of the most critical  
448 locations and assessment of the remaining service life of the studied details. This is a key step of the  
449 proposed methodology and requires a precise study of the structure and details in question. In the  
450 case of old steel structures, it is often very difficult to meet the requirements of existing standards.  
451 This may introduce additional difficulties to the assessment process.
  - 452 • As assembly details are the most critical components in fatigue, the applied reinforcement is highly  
453 localized. This allows for use of adhesively bonded reinforcement processes that require post-  
454 curing. Although such an operation was successfully applied here at real scale, there may be  
455 alternative post-curing systems which could improve the application (reduce complexity and cost).  
456 It could also be interesting to keep developing an adhesive which does not need post curing, or  
457 define the boundary conditions where the post-curing would not need to be necessary. The  
458 efficiency of stress transfer was demonstrated through the strain measurements.
  - 459 • There seems to be no effect for the studied case when reinforcing the critical detail under traffic  
460 loads. This should be more deeply studied, especially if higher service strains are encountered, but  
461 offers interesting perspectives in terms of usage constraints, avoiding traffic deviation and closures  
462 (Orcesi et al. 2019).
  - 463 • The strain measurement allowed for verification of the efficiency of the reinforcement system for  
464 both static and dynamic loading and for the three different reinforcement configurations. A 20 to  
465 30% stress decrease was accomplished, allowing for an increase in the remaining service life for the  
466 most critical detail (more than double). The measured stress decrease was slightly higher than what  
467 was expected. This may be due to local bending effects.
- 468 In addition, the onsite strain measurement resulted in noting CFRP strains were much smaller than  
469 the design strains determined through preliminary laboratory investigations.

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471

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485

486 **Data availability**

487

488 Some or all data, models, or code generated or used during the study are available from the  
489 corresponding author by request (material characterization results, residual life assessment,  
490 reinforcement design, on site load test results).

491

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622 **Fig. 1.** General layout of the Jarama Bridge (dimensions in m)  
623  
624 **Fig.2.** General views of Jarama Bridge  
625  
626 **Fig. 3.** Location of the selected butt-weld splices for CFRP strengthening (dimensions in m)  
627  
628 **Fig. 4.** Flowchart demonstrating the different components of fatigue assessment questionnaire  
629  
630 **Fig. 5.** CFRP strengthening configurations for the six studied locations (dimensions in m)  
631  
632 **Fig. 6.** a) Application of the adhesive on the plate. b) Pressure application with rollers.  
633  
634 **Fig.7.** Pictures of the heating system used for post-curing operation  
635  
636 **Fig. 8.** Picture of a pair of strain gauges installed on the bridge  
637  
638 **Fig. 9.** Picture of the debonding sensor applied on the edge before application of the adhesive  
639  
640 **Fig. 10.** The three studied static positions on the bridge during the load tests (circles correspond to the  
641 studied butt weld locations)  
642  
643 **Fig. 11.** The two studied dynamic positions on the bridge during the load tests (circles correspond to the  
644 studied butt weld locations, vertical arrow corresponds to the position of the wooden plank)  
645  
646 **Fig. 12.** Data recorded during the placement of the two trucks on the right lane in the first static position  
647 before the strengthening intervention  
648  
649 **Fig. 13.** Data recorded during the dynamic load tests on the right lane before the strengthening  
650 intervention  
651  
652 **Fig. 14.** Summary of the average strain measurements carried out before and after reinforcement for  
653 both static and dynamic tests, at the six studied locations close to the weld toe (80 mm)  
654

**Table 1.** Summary of in-shop and on-site butt-welded splices in the main girders of the Jarama Bridge

Weld N°	Span	Beam	Weld			
			Element	Distance from support, in m	Site/Workshop	Status
31	1	1 (length: 16.88 m)	Bottom flange butt weld	2.06	Workshop	Fulfills
30	1	1 (length: 16.88 m)	Bottom flange butt weld	9.44	Workshop	DOES NOT fulfill
23	1	2 (length: 16.88 m)	Bottom flange butt weld	8.08	Workshop	DOES NOT fulfill
22	1	2 (length: 16.88 m)	Bottom flange butt weld	14.08	Workshop	DOES NOT fulfill
29	2	1 (length: 23.94 m)	Bottom flange butt weld	6.11	Workshop	Fulfills
28	2	1 (length: 23.94 m)	Bottom flange butt weld	10.67	Workshop	Fulfills
27	2	1 (length: 23.94 m)	Bottom flange butt weld and web butt weld (lowerpart)	12.78	Site	DOES NOT fulfill
26	2	1 (length: 23.94 m)	Bottom flange butt weld	16.34	Workshop	Fulfills
25	2	1 (length: 23.94 m)	Bottom flange butt weld	20.04	Workshop	Fulfills
24	2	1 (length: 23.94 m)	Bottom flange butt weld	23.01	Workshop	DOES NOT fulfill
21	2	2 (length: 23.94 m)	Bottom flange butt weld	1.3	Workshop	DOES NOT fulfill
20	2	2 (length: 23.94 m)	Bottom flange butt weld	7.84	Workshop	Fulfills
19	2	2 (length: 23.94 m)	Bottom flange butt weld	12.41	Workshop	Fulfills
18	2	2 (length: 23.94 m)	Bottom flange butt weld and web butt weld (lowerpart)	14.41	Site	DOES NOT fulfill
17	2	2 (length: 23.94 m)	Bottom flange butt weld	17.14	Workshop	Fulfills
16	2	2 (length: 23.94 m)	Bottom flange butt weld	19.94	Workshop	Fulfills
15	2	2 (length: 23.94 m)	Bottom flange butt weld	22.98	Workshop	Fulfills
6	4	1 (length: 23.88 m)	Bottom flange butt weld	1.05	Workshop	DOES NOT fulfill
7	4	1 (length: 23.88 m)	Bottom flange butt weld	3.08	Workshop	Fulfills
8	4	1 (length: 23.88 m)	Bottom flange butt weld	6.44	Workshop	Fulfills
9	4	1 (length: 23.88 m)	Bottom flange butt weld and web butt weld (lowerpart)	11.35	Site	DOES NOT fulfill
10	4	1 (length: 23.88 m)	Bottom flange butt weld	16.95	Workshop	Fulfills
11	4	1 (length: 23.88 m)	Bottom flange butt weld	22.98	Workshop	DOES NOT fulfill
1	4	2 (length: 23.88 m)	Bottom flange butt weld	2.75	Workshop	Fulfills
2	4	2 (length: 23.88 m)	Bottom flange butt weld	5.86	Workshop	Fulfills
3	4	2 (length: 23.88 m)	Bottom flange butt weld and web butt weld (lowerpart)	9.57	Site	DOES NOT fulfill
4	4	2 (length: 23.88 m)	Bottom flange butt weld	14.68	Workshop	DOES NOT fulfill
5	4	2 (length: 23.88 m)	Bottom flange butt weld	20.14	Workshop	DOES NOT fulfill
14	5	1 (length: 16.63 m)	Bottom flange butt weld	8.18	Workshop	DOES NOT fulfill
12	5	2 (length: 16.63 m)	Bottom flange butt weld	6.08	Workshop	DOES NOT fulfill
13	5	2 (length: 16.63 m)	Bottom flange butt weld	11.99	Workshop	Fulfills

**Table 2.** Summary of key fatigue design variables used in FASSTbridge fatigue assessment methodology tool

Description	AASHTO	Eurocode
Fatigue loads	Live Load ( <i>LL</i> ), Dynamic Load ( <i>IM</i> ), Centrifugal Load ( <i>CE</i> )	Fatigue Load Model 1, 2 or 3
Average daily traffic	<i>ADT</i>	n/a
Number of trucks in traffic	Highway classification ( <i>h</i> )	Traffic category based on $N_{obs}$ /per year ( $\lambda_2$ )
Average daily truck traffic	<i>ADTT</i> (= <i>ADT</i> x <i>h</i> )	Traffic type
Fraction of truck traffic in a single lane	Number of lanes available to trucks ( <i>p</i> )	Number of slow lanes
Truck traffic in a single lane	$ADTT_{SL}$ (= <i>ADTT</i> x <i>p</i> ) / per day	$N_{obs}$ /per year
Stress range from truck passage	Cycles per truck passage ( <i>n</i> )	Damage effect of traffic ( $\lambda_f$ )
Detail design constant	Detail category ( <i>A</i> )	Detail category ( <i>CAFT</i> for $N=2000000$ )
Stress range cycles over design life	Number of cycles ( <i>N</i> )	Number of cycles ( <i>N</i> )
Recommended design life	75 years	100 years

**Table 3.** Proposed modification factors ( $\alpha$  values) used for the (a) AASHTO and (b) Eurocode fatigue assessment and remaining fatigue life calculations.

$a_n$	AASHTO	Eurocode
$a_1$	$A$ : design detail fatigue resistance	$\Delta\sigma_c$ : design detail fatigue resistance
$a_2$	$N$ : number of stress range cycles	$\gamma_{Mf}$ : partial factor for fatigue strength
$a_3$	$ADT$ : average daily traffic	$\lambda_1$ : damage effect of traffic
$a_4$	$p$ : single lane	$N_{obs}$ : single lane truck traffic
$a_5$	Single lane truck traffic	$\lambda_2$ : expected annual traffic volume
$a_6$	$[(\Delta f)_{eff}]^3$ : fatigue load stress range	$\gamma_{FF}$ : safety factor for fatigue loading
$a_7$	Girder type, effects of corrosion, level of importance, risk tolerance, etc ...	$\Delta\sigma_{FLM3}$ : fatigue load stress range

**Table 4.** Remaining fatigue life of studied assembly at the Jarama Bridge

Weld No	Weld Code	Remaining service life
29	S2-B1-W1	> 100 years
26	S2-B1-W4	> 100 years
25	S2-B1-W5	> 100 years
20	S2-B2-W2	40 - 50 years
17	S2-B2-W5	> 100 years
16	S2-B2-W6	> 100 years

**Table 5:** Strengthening configuration used in selected welds.

Weld No.	Weld code	Number of laminates	Number of layers	Length [mm]		
				Layer 1	Layer 2	Layer 3
29	S2-B1-W1	4	2	1400	1200	-
26	S2-B1-W4	5	1	1200	-	-
25	S2-B1-W5	3	3	1600	1400	1200
20	S2-B2-W2	4	2	1400	1200	-
17	S2-B2-W5	5	1	1200	-	-
16	S2-B2-W6	3	3	1600	1400	1200

**Table 6.** Estimated remaining fatigue life of specific joints at the Jarama Bridge, before and after strengthening

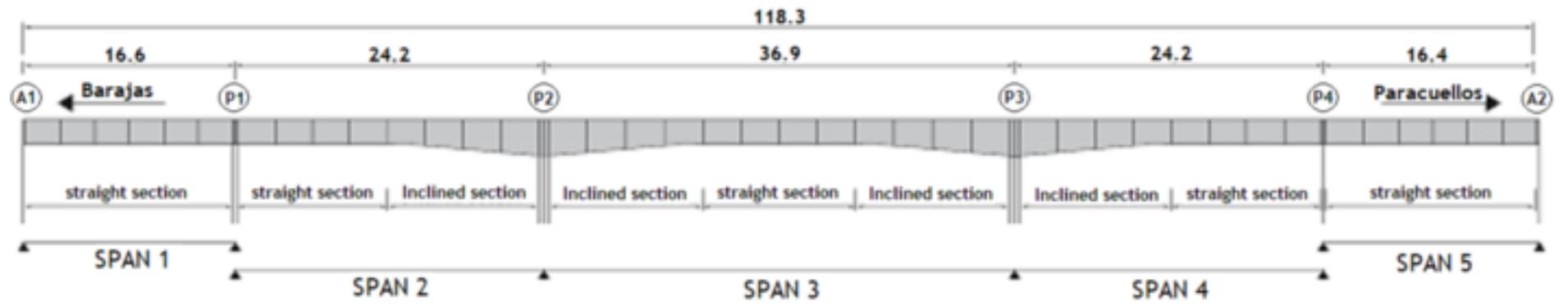
Weld No.	Weld code	Remaining Fatigue Life (years)	
		Unstrengthened Girders	CFRP Strengthened Girders
29	S2-B1-W1	> 100 years	> 100 years
26	S2-B1-W4	> 100 years	> 100 years
25	S2-B1-W5	> 100 years	> 100 years
20	S2-B2-W2	40 - 50 years	120 - 130 years
17	S2-B2-W5	> 100 years	> 100 years
16	S2-B2-W6	> 100 years	> 100 years

**Table 7.** Maximum measured differential strains before and after reinforcement for static and dynamic load tests and for the six studied locations close to the weld toe (80 mm)

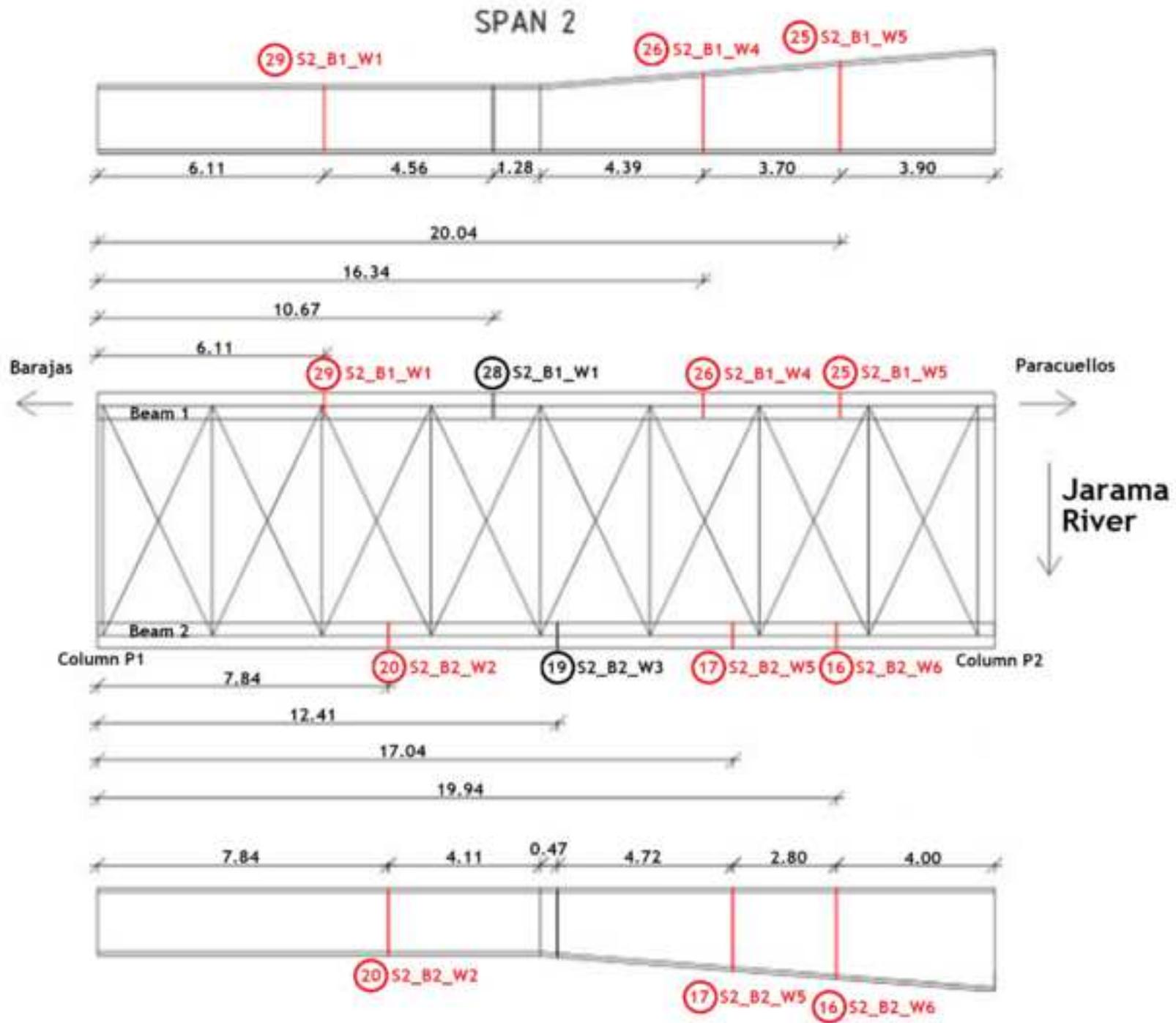
Sensor	Laminates	Static before	Static after	Difference	Dynamic before	Dynamic after	Difference
29-1	2 x 4 (8)	77 $\mu\epsilon$	58 $\mu\epsilon$	- 25 %	71 $\mu\epsilon$	51 $\mu\epsilon$	- 28 %
26-1	1 x 5 (5)	84 $\mu\epsilon$	67 $\mu\epsilon$	- 20 %	84 $\mu\epsilon$	77 $\mu\epsilon$	- 8 %
25-1	3 x 3 (9)	69 $\mu\epsilon$	51 $\mu\epsilon$	- 26 %	65 $\mu\epsilon$	53 $\mu\epsilon$	- 18 %
20-1	2 x 4 (8)	84 $\mu\epsilon$	59 $\mu\epsilon$	- 30 %	80 $\mu\epsilon$	56 $\mu\epsilon$	- 30 %
17-1	1 x 5 (5)	68 $\mu\epsilon$	56 $\mu\epsilon$	- 18 %	75 $\mu\epsilon$	62 $\mu\epsilon$	- 17 %
16-1	3 x 3 (9)	71 $\mu\epsilon$	56 $\mu\epsilon$	- 21 %	60 $\mu\epsilon$	45 $\mu\epsilon$	- 25 %

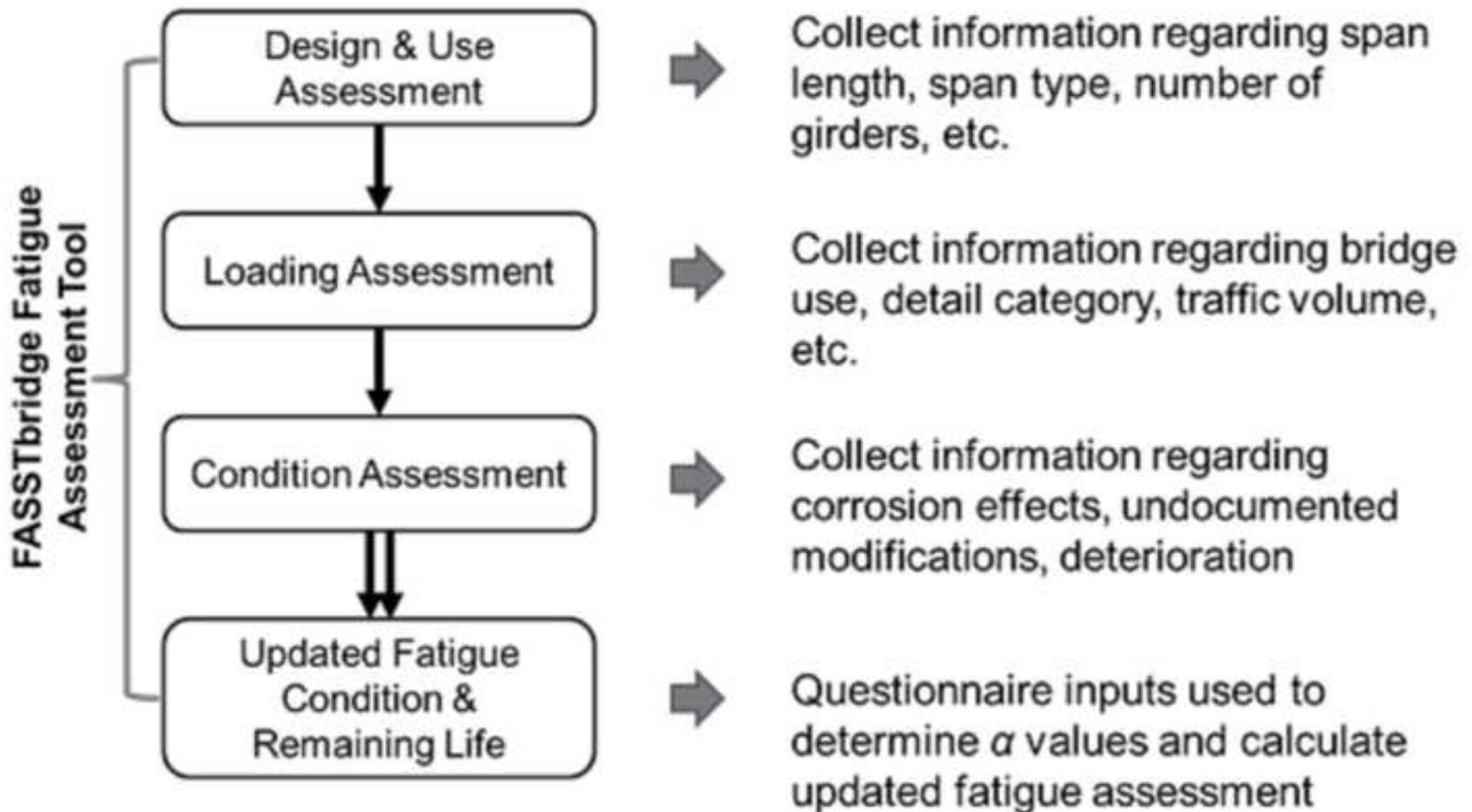
**Table 8.** Maximum measured differential strains before and after reinforcement for static and dynamic load tests and for the six studied locations far from the weld toe (950 mm)

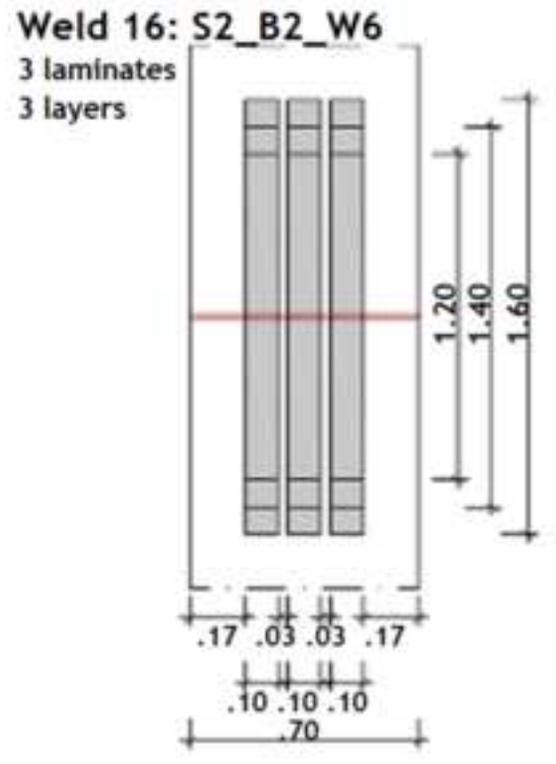
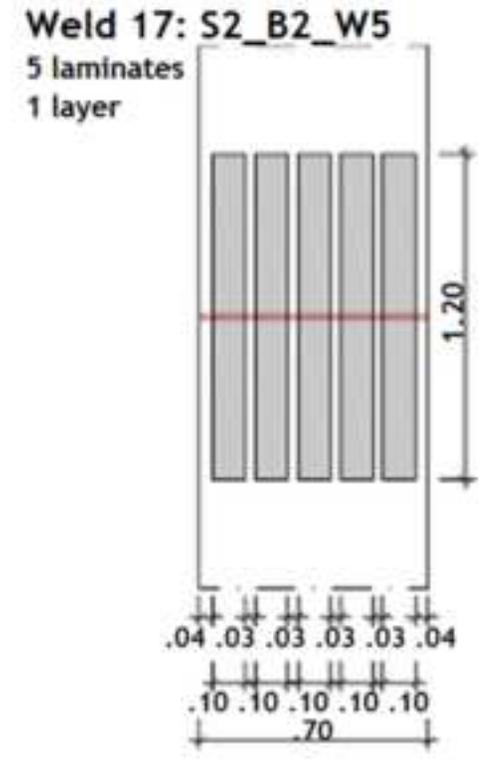
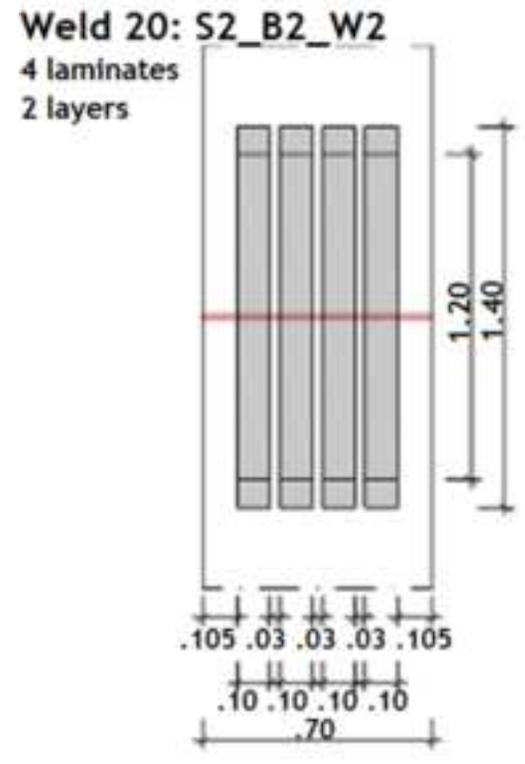
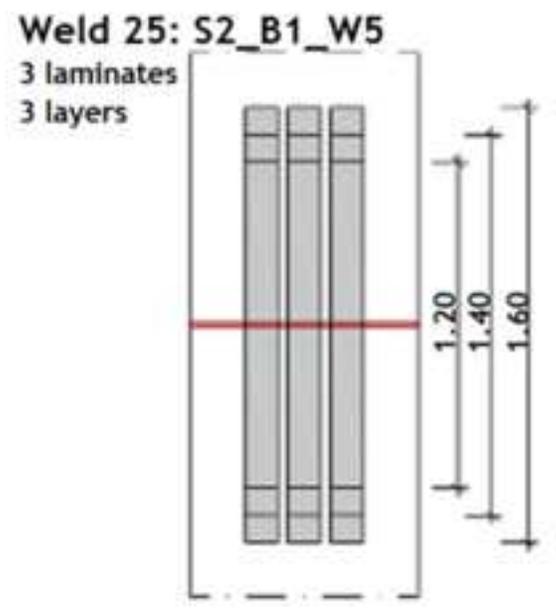
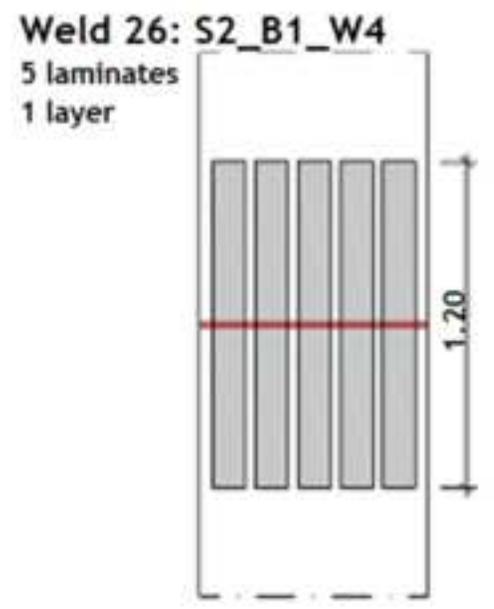
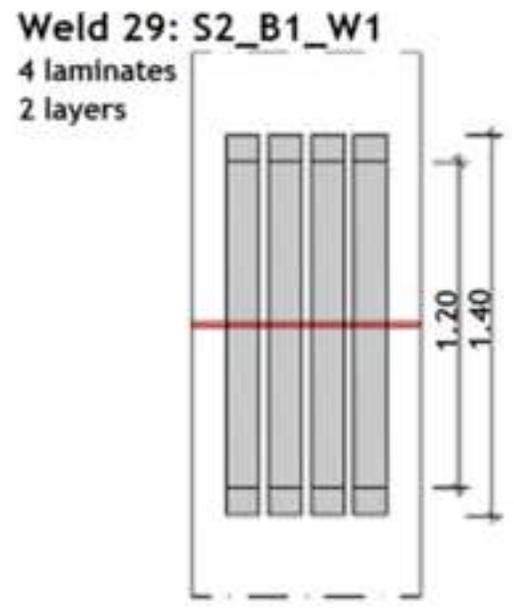
Sensor	Laminates	Static before	Static after	Difference	Dynamic before	Dynamic after	Difference
29-2	2 x 4 (8)	71 $\mu\epsilon$	63 $\mu\epsilon$	- 11 %	64 $\mu\epsilon$	61 $\mu\epsilon$	- 5 %
26-2	1 x 5 (5)	85 $\mu\epsilon$	72 $\mu\epsilon$	- 15 %	91 $\mu\epsilon$	88 $\mu\epsilon$	- 3 %
25-2	3 x 3 (9)	63 $\mu\epsilon$	55 $\mu\epsilon$	- 13 %	60 $\mu\epsilon$	55 $\mu\epsilon$	- 8 %
20-2	2 x 4 (8)	77 $\mu\epsilon$	66 $\mu\epsilon$	- 14 %	74 $\mu\epsilon$	62 $\mu\epsilon$	- 16 %
17-2	1 x 5 (5)	82 $\mu\epsilon$	71 $\mu\epsilon$	- 13 %	92 $\mu\epsilon$	80 $\mu\epsilon$	- 13 %
16-2	3 x 3 (9)	69 $\mu\epsilon$	59 $\mu\epsilon$	- 14 %	60 $\mu\epsilon$	52 $\mu\epsilon$	- 13 %













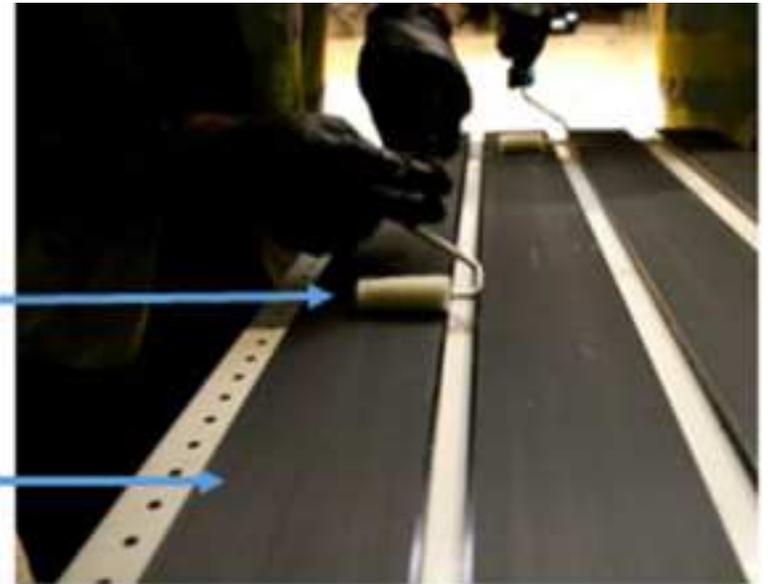
a)

*Adhesive application*

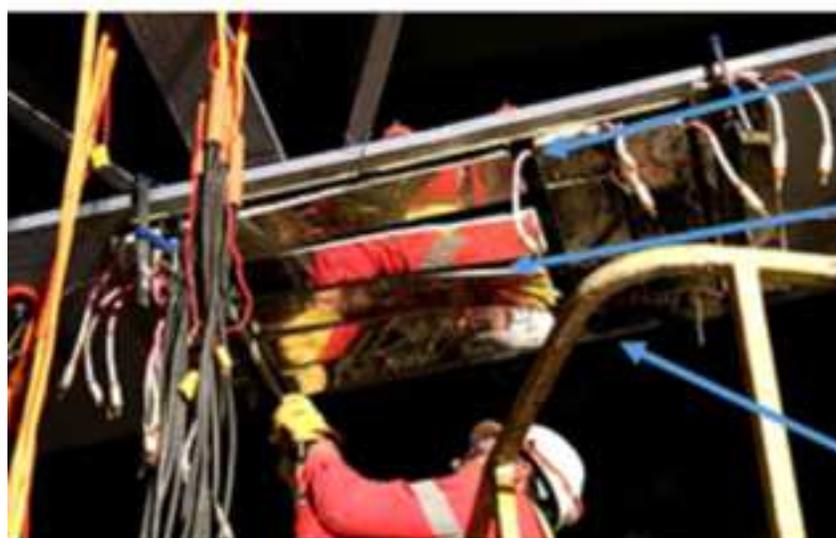
*Thickness control*

*Roller to apply pressure*

*CFRP plate*



b)



*Adhesively bonded CFRP*

*Heating pads*

*Insulation*

*Steel structure*





*Steel  
bottom  
flange*

*Temperature  
compensation*

*Connection to  
acquisition*

*Strain  
measurement*

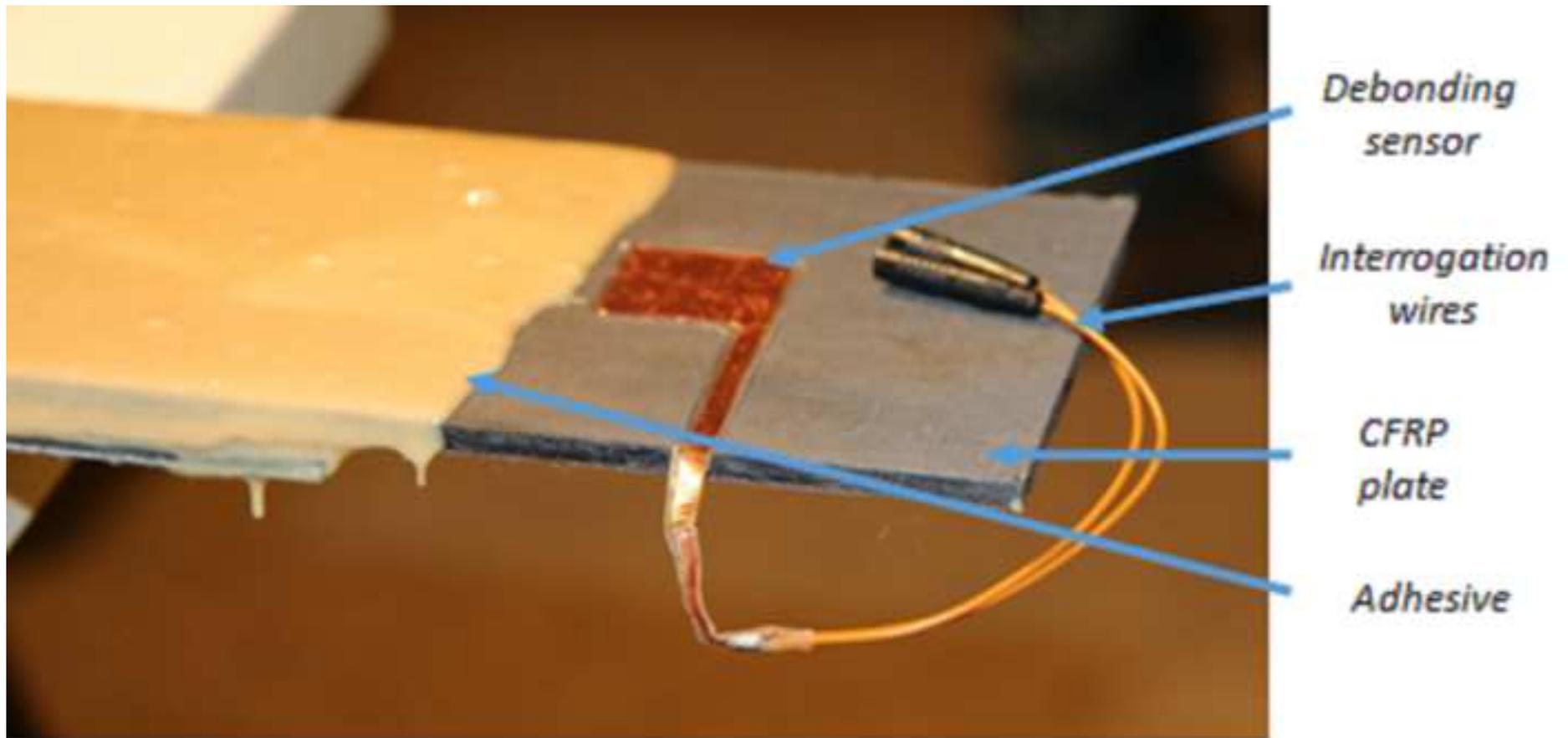


Figure 10

