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PREVENTIVE FATIGUE STRENGTHENING OF STEEL STRUCTURES WITH BONDED CFRP – EFFICIENCY DEMONSTRATION ON A REAL BRIDGE

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ABSTRACT: One of the goals of sustainable development applied to bridge infrastructure is to provide bridge owners with strengthening solutions that may lead to an increase of existing structures service life. In the case of steel bridges, the assessment of the remaining service life is most often linked to the determination of the structural deterioration caused by corrosion and fatigue. Damage caused by fatigue is very difficult to assess before crack initiation, and is more bound to occur in old structures, for which the phenomenon was not taken into account in design before 1970. In addition, old steel materials present a more brittle behavior. To address these issues, a preventive methodology for fatigue strengthening of steel structures and a strengthening solution based on the use of an adhesively bonded UHM (Ultra High Modulus) CFRP (Carbon Fiber Reinforced Polymer) plate was developed and applied to an existing bridge (Jarama bridge near Madrid in Spain) for validation. Strain measurement was carried out to verify the theoretical expectations of the reinforcement, and different parameters were studied such as the influence of low traffic during reinforcement application. The obtained results proved the efficiency of the system for the studied structure.

1. Introduction

Steel bridges present frequent fatigue damages that reduce their load-carrying capacity and residual life. The increase of traffic flows and loads over the last decades has emphasized this problem, especially on ageing structures. In Europe, 15% of the 300 000 bridges are made of steel or concrete-steel structures, and approximately 68% of them need structural interventions. In the USA, figures are quite similar.

Having this in mind, in November 2015, the ERA-NET Plus project FASSTBRIDGE was launched aiming at developing a complete methodology including a calculation method to assess the remaining life of steel bridge in terms of fatigue, a design and application method for the strengthening intervention, and a strengthening system relying on the use of adhesively bonded UHM-CFRP plates (Ultra High Modulus Carbon Fiber Reinforced Polymer Composites).

The paper presents the FASSTbridge methodology and strengthening system, which were designed with codes based both on The American Association of State Highway and Transportation Officials (AASHTO) or the European Union's Eurocode, and their implementation in Jarama Bridge (Madrid, Spain).

2. Description of the Bridge selected for the demonstration (Jarama Bridge)

Jarama Bridge is located in Madrid (Spain) over Jarama river on road M-111. It was designed in 1962 (Design Code 1956) and erected around 1965. It is a 3 span continuous beam with two lateral simply supported spans, with the following distribution: 16.6 m – 24.2 m – 36.9 m – 24.2 m – 16.95 m (Figure 1.a). The total length is 118.85 m. The main structure is made of structural steel whereas the top slab is in reinforced concrete with a width of 9,20 m. The whole structure is welded. The cross-section consists of two I-beams connected by transversal cross-bracings to stiffen the bridge in transversal direction (Fig. 1.b). The design code of 1956 did not take into account any fatigue limit state. Adequacy of the structure for the project needed to be checked by carrying-out a fatigue limit state analysis.

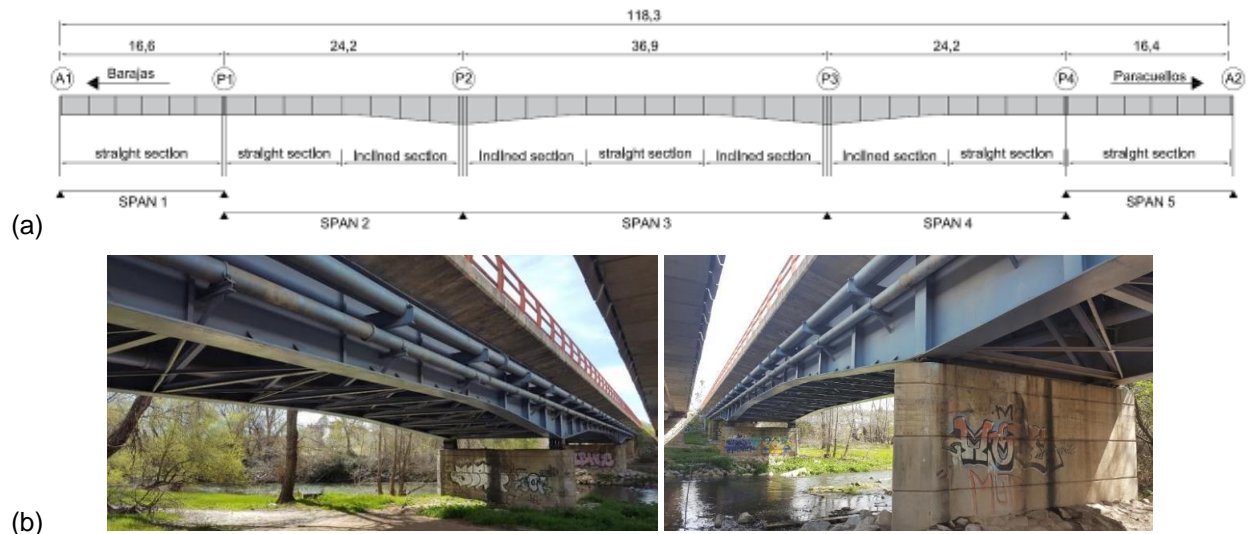


Fig. 1 – Span distribution (a) and general views (b) of Jarama bridge

3. Diagnosis of Jarama bridge

3.1. Condition of the bridge

A detailed inspection of the structure was carried out to determine its adequacy for the demonstration. As the project is dedicated to preventive intervention, the structure must be in good condition for the retrofit and free from fatigue damages. Steel elements in Jarama Bridge were generally in good condition. Still, there was moisture, moss, corrosion and some other local detectable defects. On the top flange of the main girders, general corrosion was notable. In some cases, corrosion had already progressed into the web plates. In some other cases, efflorescence and peeling problems were noted. The same deficiencies were also present on the bottom flange. Crosses or x-bracing seem in good condition and no visual defect was

noted. Nevertheless, bottom lateral bracing members of span 1 show important distortion, corrosion and some other defects probably caused by object impact. One angle in bottom lateral bracing member of span 5 also presented small deformation.

The floor beams were also in good condition. Moisture, efflorescence and corrosion on the top flange were locally noticed. Some areas of peeled coating can be seen in the bottom flange. Larger areas affected by moisture and corrosion are located in floor beams over piers and abutments.

3.2. Selection of welds for the intervention

There were two types of butt welds in the main girders: some were welded on-site and other at workshop. During the second bridge-inspection in 2017, four out of the 8 on-site welds were assessed. In addition to visual inspections, ultrasonic tests were carried out, which showed that the selected welds did not fulfil quality standards according to level B UNE EN ISO 5817:2014 neither AL2 UNE EN ISO 11666:2011. From the 27 welds of the second type (in-shop-welds) that were analysed, eleven did not fulfil the quality standards (under AL2 UNE EN ISO 11666:2011).

In order to meet the project budget, accessibility and installation cost must be taken into account. Main span over the river may be the most interesting to strengthen from a structural point of view, but that would have resulted in budget overspending. It was thus decided to focus on side spans of the bridge.

Span number 2 was selected for the demonstration (Barajas side, see Fig. 1.a). Single beams of span 2 are named "beam 1" and "beam 2". On each beam, four butt welds were worth investigating. Two were discarded because they are too close to the kink of the bottom flange. Therefore, only welds n° 29, 26, 25, 20, 17 and 16 were investigated (Fig 2.a) and effect of strengthening action was evaluated. In order to get as much information as possible from the demonstration, it was decided to apply different CFRP plate layer arrangements at these selected weld locations: one single layer of 5 plates, 2 layers of 4 plates, and 3 layer of 3 plates.

4. Theoretical calculations and design of intervention

4.1. FEM modeling

To evaluate the strengthening action of CFRP laminates on the butt welds of span 2, a 3D-shell-model was created in FEM-Program (Fig. 2). The two main girders are modelled with shell elements. The deck slab carries out the connection between both girders. For the modelling, the slab induces no loadings into the system and the stiffness in longitudinal direction was set to zero. This allows ensuring a realistic load distribution on both of the girders. The stiffness in transversal direction is achieved in reality by cross-bracings. To simplify the model, these latter are idealised by bearings fixed in y-direction.

To represent the welds, the height of the bottom flange was reduced on a small segment with a width of 1-2 cm. It is then possible to get precise results on the stresses with and without strengthening action.

4.2. Evaluation of strengthening action

Calculations were conducted, in order to derive the algebraic difference between the minimum and maximum stresses in the welds due to fatigue loading (noted $\Delta\sigma_{fat}$). Having this value of $\Delta\sigma_{fat}$, an evaluation of the fatigue strength is possible.

Beforehand several scenarios were investigated to estimate an interaction between the welds (possible influence on the neighbouring weld if only one of the welds of the beam is strengthened). From the results of this preliminary investigation, it was decided to strengthen all 6 welds as shown in Fig. 2.b.

For the six welds loaded with tensile stresses, the reduction of $\Delta\sigma_{fat}$ was estimated. As expected, there is a reduction of $\Delta\sigma_{fat}$ due to the amount of CFRP laminates and layers. The more laminates next to each other are applied, the higher is the decrease in $\Delta\sigma_{fat}$. The same occurs with more layers of laminates. The option "2 layers of 4 plates" per weld provided a reduction of $\Delta\sigma_{fat}$ of about 8 - 15 %, while option "1 layer of 5 plates" gave reduction in the range 4 - 7 %, and option "3 layers of 3 plates" led to a reduction in the range 21 - 22 %. More details can be found in Chataigner et al (2020).

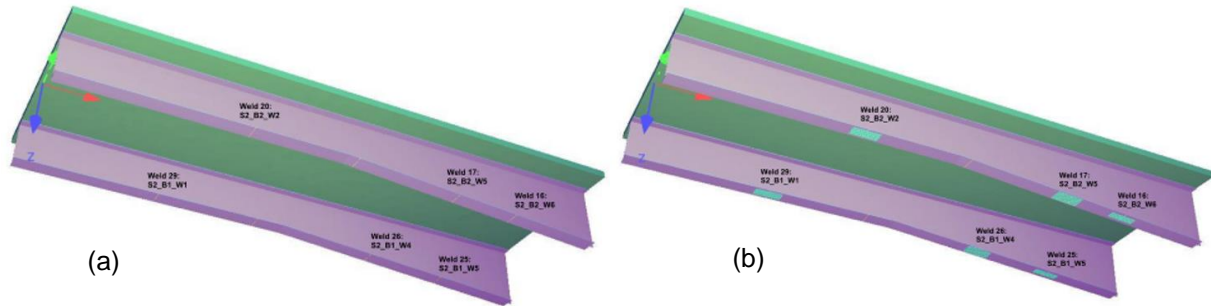


Fig. 2 – Models of the studied span of Jarama bridge (Span 2) before (a) and after (b) reinforcement at the 6 selected weld locations.

5. Implementation of the strengthening solution in the field

5.1. Preparation of UHM-CFRP plates

Ultra-high modulus CFRP plates (460,000 N/mm², 66,700 ksi) were provided by Epsilon Composite (France). They were cut to the required length and their ends detailed as specified by the designer prior to the preparation of the surfaces for bonding. In addition, when several layers of CFRP reinforcement were considered, the adhesive bonding of the CFRP layers was done at the workshop before being delivered on site (this minimizes the time and the number of operations on site).

5.2. Steel surface preparation

In FASSTbridge solution, no primer is needed provided the adhesive application is made shortly after steel surface preparation. A control of the surface preparation quality was done before application of the adhesive. The level of surface preparation by sanding was determined using NF EN ISO 8501-1 (a minimum value of 2.5 is required). Roughness was controlled using NF EN ISO 8503-2. Surface cleanliness was also assessed using NF EN ISO 8502-3.

5.3. Adhesive mix and application

A hybrid two component epoxy-polyurethane adhesive (Urepox Extra 2C) was used, with the following specifications: (1) Pot life 30-45 minutes (workability); (2) glass transition temperature $T_g > 71^\circ\text{C}$ after post-curing; (3) Tensile strength ≥ 35 MPa, 5,1 ksi. This adhesive was developed specifically for this project by Collanti Concorde (Italy). The only way of achieving the target T_g of 71°C (fixed at the beginning of the project), was to apply a post-curing procedure at high temperature (1 hour at 80°C) as detailed in (Chataigner, 2018). Implementing this post-curing process on site was quite challenging.

The adhesive mix was applied directly to the surface of the CFRP plate. In general, the target thickness for metal and FRP joints should be 0,5-2,0 mm. To spread the adhesive uniformly, a trowel with a v-notch was used (Fig. 3.a). The application was carried out on CFRP surface, and the impregnated plate was then applied to the steel surface; a roller was used to apply pressure along the reinforcement and remove voids from the adhesive layer (Fig. 3.b).



Fig. 3 – CFRP preparation: (a) Application of the adhesive layer and (b) pressure application

5.4. Installation in the field and post-curing process

Post-curing (1h at 80°C) of the bonded CFRP system attached to the structure was achieved onsite using ceramic pad units, isolating blankets and a clamping device (Fig. 4). Temperature at the surface of the CFRP plate was monitored continuously during the whole process, in order to control the heating rate and the target post-cure temperature. After removal of the heating system, paint was applied on the whole reinforcement and surrounding steel to protect the area where the coatings had been removed.

Each of the six weld locations was reinforced during night to reduce traffic disruption. For 3 weld locations, traffic was partially deviated during the whole application (one carriageway instead of 2 carriageways). For 2 locations (welds n°26 and 29), traffic was deviated only during the application of the bonded CFRP plates before post-curing. For one location (weld n°16), traffic was not deviated at all.

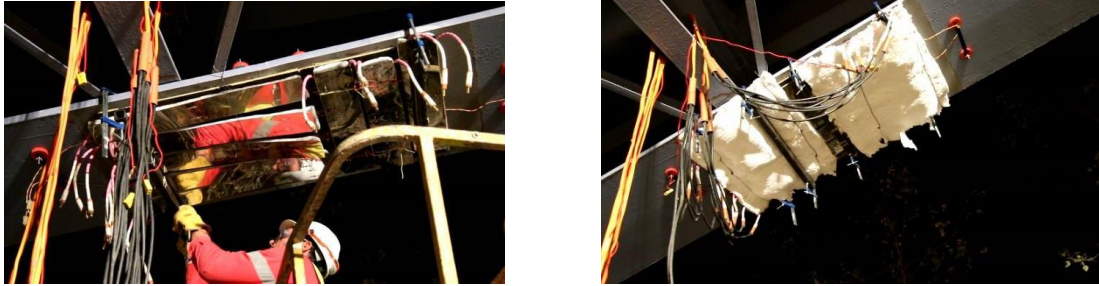


Fig. 4 – Installation of the heating system used for post-curing operation

6. Effectiveness of the proposed solution – Results of load tests

In order to gain knowledge about the condition of the Jarama bridge, a fixed load test was performed. This fixed load condition test allowed, by measuring strains before and after retrofitting, the demonstration of the effectiveness of the intervention. The tests took place during two nights before and after the retrofitting. All the load tests were repeated for both lanes during one-sided traffic blocking of the bridge.

Static tests were first performed by placing two 27 tons trucks (Renault Kerax) at three predefined positions of the bridge. Dynamic load tests were also performed by placing a wooden plank, 4 cm in height, at two selected positions, over which a single truck passed at a speed of 20 km/h and 50 km/h. The passage of the truck when hitting with plank, induced a dynamic impact load in the structure. Fig 5. shows examples of positioning of the truck for both static and dynamic load tests.

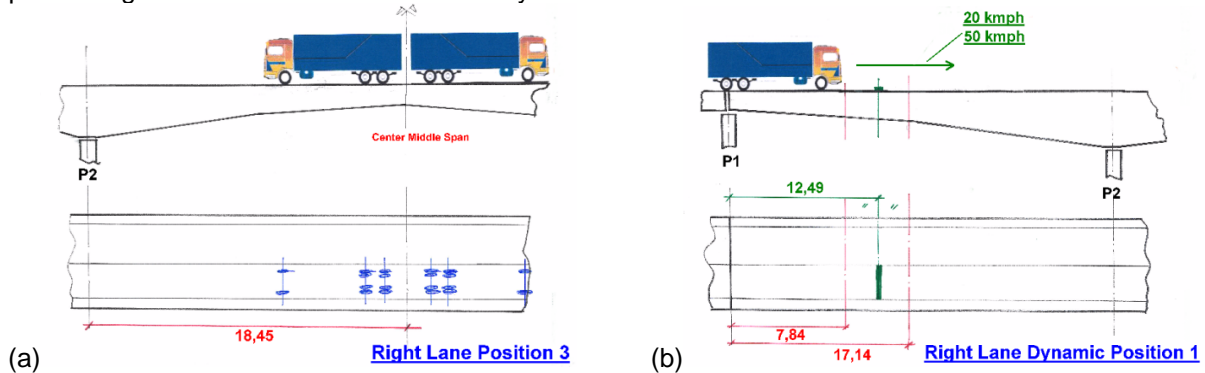


Fig. 5 – Examples of positioning of the trucks during static (a) and dynamic (b) load tests

Main results collected by strain gages at the six weld locations are given in Table 1 and in Fig. 6 before and after reinforcement and for both static and dynamic tests. In all cases, a strain decrease between 8 and 30 % was observed after retrofitting, in agreement with theoretical expectations. The highest strain reductions (20 to 30 %) were obtained for locations reinforced with the highest numbers of CFRP plates, showing that the proposed solution works well in the case of the 3 layer configuration (welds 16 and 25). In addition, there was no discrepancy (in terms of strain reduction) for the locations that were reinforced without total traffic disruption (weld n° 16, 26 and 29) compared to those reinforced with total disruption. This suggests that the solution could be applied under low traffic, which would be a major advantage.

Table 1 - Maximum measured differential strains before and after reinforcement for static and dynamic load tests and for the six studied locations close to the weld toes (80 mm)

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

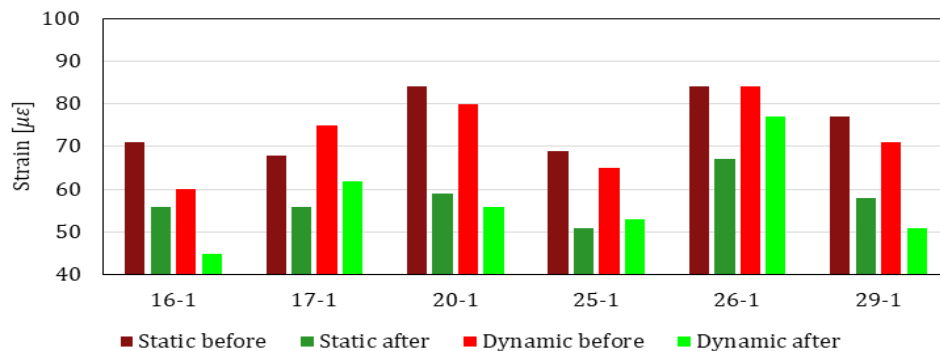


Fig. 6. Strain measurements during static and dynamic load tests before and after retrofitting

7. Conclusions

The results of FASSTbridge project presented in this article allows proposing a methodology to extend service life of existing steel bridges prone to fatigue loading based on the following stages: Diagnosis, Safety Evaluation, Design of Intervention, Monitoring and Maintenance, in a prefailure scenario. In this line, a fatigue assessment tool was developed for existing structures. Depending on diagnosis, a retrofit design can be proposed if needed, which relies on the use of an adhesively bonded UHM- CFRP plate.

FASSTbridge methodology was demonstrated on a real structure (Jarama bridge) near Madrid (Spain). A complete diagnosis of the bridge was carried out, and the remaining service life assessed using the developed methodology. The proposed reinforcement was applied on six locations to check the ease of application and the efficiency of the method through strain monitoring before and after reinforcement during controlled on site load tests. The application was successful and allowed fine-tuning of the solution and proposing improvements for similar operations. More details can be found at www.fasstbridge.eu.

8. Acknowledgements

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9. References

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