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Extending the lifetime of steel truss bridges by cost-efficient strengthening interventions

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ABSTRACT

The large number of existing bridges and viaducts all around the country has become a major problem for bridge owners. In the specific case of steel truss bridges, a reasonable method to extend the lifetime of existing structures is represented by the introduction of new deck systems combined with diffused strengthening interventions. In this study, a stepwise approach considering different loading conditions is presented (historical and Eurocode loads). The structural analysis has been performed on a case study bridge with a finite element model (FEM) calibrated on load tests. It was found that the predicted deformation agreed reasonably with the experimental results. Different strengthening alternatives were analysed and discussed: the introduction of orthotropic deck; the construction of composite deck with differentiating thickness and ordinary concrete strength; the construction of composite deck with differentiating thickness and high concrete strength; in some cases, also steel-to-steel interventions on the bridge are provided. It has been found that the best structural strengthening alternative lies in the construction of a composite concrete or of an ultra high-performance concrete (UHPC) or an ultra high-performance fibre-reinforced concrete (UHPFRC) deck with a reduced thickness (compared with traditional interventions) resting on the existing steel structure combined with steel-to-steel interventions.

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Composite structures; assessment; strengthening; steel trusses; cost optimisation; steel bridges

1. Introduction

Bridges are a strategic part of an ancient transport network and, in some cases, they are at the limits of the traffic capacity. In the particular case of steel bridges, truss bridges were widely used during road construction from the second half of the nineteenth century up to the middle of the twentieth century. Most of these wrought-iron or older steel bridges, which are still in use around Europe, were not designed explicitly for continuously increasing vehicles numbers and weight. ASCE (1982) reported that 80–90% of failures in steel structures are related to fatigue and fracture. However, other factors affecting the structural ageing of bridges are reported by Bruhwiler et al. (1990), Kulak (1992), Akesson and Edlund (1996), Di Battista et al. (1997), Bursi et al. (2002), Matar and Greiner (2006), Pipinato (2008), Pipinato et al. (2009, 2011), Boulent et al. (2008), Albrecht and Lenwary (2008, 2009). Vibrations, transverse horizontal forces, internal constraints, localised and diffused defects as corrosion damages, are concurring causes of damages (Byers et al., 1997).

The main problems recognised by the managing agencies are related to difficulties in maintenance, high noise emissions and vibrations, fatigue and understrength capacities mainly found in transverse, main girders and their riveted or bolted connections while the main load bearing elements (trusses) still have some residual capacity (Pipinato et al., 2009, 2011). Another major problem is the inability to carry the actual Eurocode live loads (EN 1993-1-1, 2005). It should be noticed that rarely a load reduction is approved for road of national importance in Italy (highway and national road); and for this reason existing bridges must carry the traffic category requested for new bridges (EN 1993-1-1, 2005; Italian Ministerial Decree, 2008).

Actually, a total replacement of these bridges is not possible, due to financial constraints. Moreover, most of these bridges have not yet fulfilled their design life, and in some cases their main structures are in a good condition, except specific understrength members. However, it should be mentioned that no design life was ever defined for the large majority of existing bridges which means that implicitly they are supposed to last as long as the utilisation (f.ex. for road traffic) is given.

For this reason, it is crucial to implement strengthening solutions that can extend the life of existing steel bridges, especially considering the introduction of a new deck system (orthotropic or composite steel–concrete) able to cope with the actual code requirement EN 1993-2 (2006). Fibre reinforcing solutions are not investigated in this study, even though a considerable amount of work has been done in this field (e.g. Ghafoori & Motavalli, 2015). The proposed strengthening scheme includes: (a) a new orthotropic or composite steel–concrete (ordinary or high strength) deck, combined with (b) steel-to-steel interventions. While traditional strengthening of trusses implies the use of a composite deck that is commonly of 200 mm, that implies a noticeable added dead load onto the deck, the proposed strengthening alternatives on existing steel trusses help in reducing the

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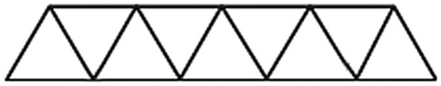
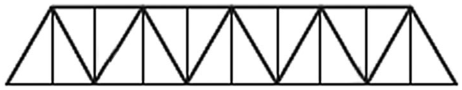
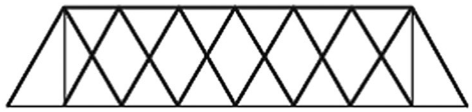
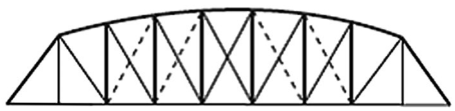
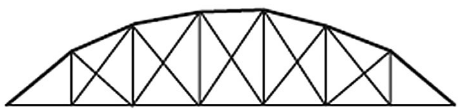
Table 1. Common typologies of truss structures analysed in the project.

Designation	Geometric scheme	Built starting from	Typical length
Warren	 or 	1848	15–120 m
Double intersection Warren		Mid nineteenth century	23–120 m
Parker		Mid to late nineteenth century	12–75 m
Bowstring arch-truss		1840	15–250 m

added weight needed for the strengthening intervention, redistributing at the same time internal forces in each type of truss members, optimising costs and time saving.

2. Typologies of truss bridges

A particular type of bridge is the truss, which is typically made entirely of steel. Trusses are assumed to be pin-jointed. This assumption means that members of the truss (chords, verticals, and diagonals) will act only in tension or compression. A more complex analysis is required where rigid joints impose significant bending loads upon the elements, as in a Vierendeel truss. A wide range of truss types have been developed, each with a special use. Many variations on these common schemes can be found in the literature. In Table 1, the most common truss types are listed: it should be clearly stated that the strengthening intervention analysed in this research refers most to truss typologies as the Warren and Parker type, while for bowstring arch-truss different interventions (e.g. including post-tensioned cables) should be considered.

3. Case study

The investigated structure is a two-lane roadway steel bridge truss. The overall bridge length is about 120 m through three spans (40 m each one). Simple truss girders at a distance of 7.6 m are simply supported on the abutment and on two central piles in the river bed. The superstructure consists of riveted built-up truss members. Lower chords are inverse T-shaped sections, diagonals and upper chords are C-built-up elements with battens (stiffening brackets), while struts are I-section shaped built-up elements composed of four L-shaped elements and a plate. The deck is built with longitudinal stringers and transverse floor beams. The floor beams have a fixed distance of 4 m, while the stringers are at 1.15 m one to the other. Top and bottom double-L bracings provide adequate stiffening of the structure. Built-up members, are built with plates, L-profiles or C profiles, are connected by hot riveting and connection joints are made of gusset riveted plates. The main structure (Figures 1 and 2) and the member details (Figure 3) have been completely revealed by an on-site geometrical survey performed in 2015–2016.

4. Structural analysis

4.1. Material parameters

Material properties knowledge of existing metal bridges is essential for the assessment of the remaining lifetime. For old metal bridges built between 1870 and 1940, the material parameters are commonly not available and material tests are recommended to evaluate steel properties such as strength and toughness. Furthermore, it has to be mentioned, that although the development of the steel grades increased during the first decades of the twentieth century, the quality of the steels themselves might be low especially during the years of the First World War (1914–1918), the great depression (1929–1939) and during and after the Second World War (1939–1950): steel production had to be fast, and expensive alloys were not available. However, twentieth-century low-alloyed mild steel basically has a homogeneous small-grained microstructure and quite good mechanical properties that make it compatible with current S275 (Kühn et al., 2008). According to historical original design documents, the following mechanical properties have been defined: $Re \approx 240$–280 N/mm^2 (yield strength), $Rm \approx 370$–450 N/mm^2 (tensile strength), $\varepsilon \approx 15$–25% (strain). According to modern standards (EN 10025, 2005), the steel can be classified as a weldable S275 steel (EN 1993-2, 2006).

4.2. Geometrical sections and structure degradation

The existing structure is not severely damaged by corrosion. Occasionally, maintenance works after steel bridge reconstruction

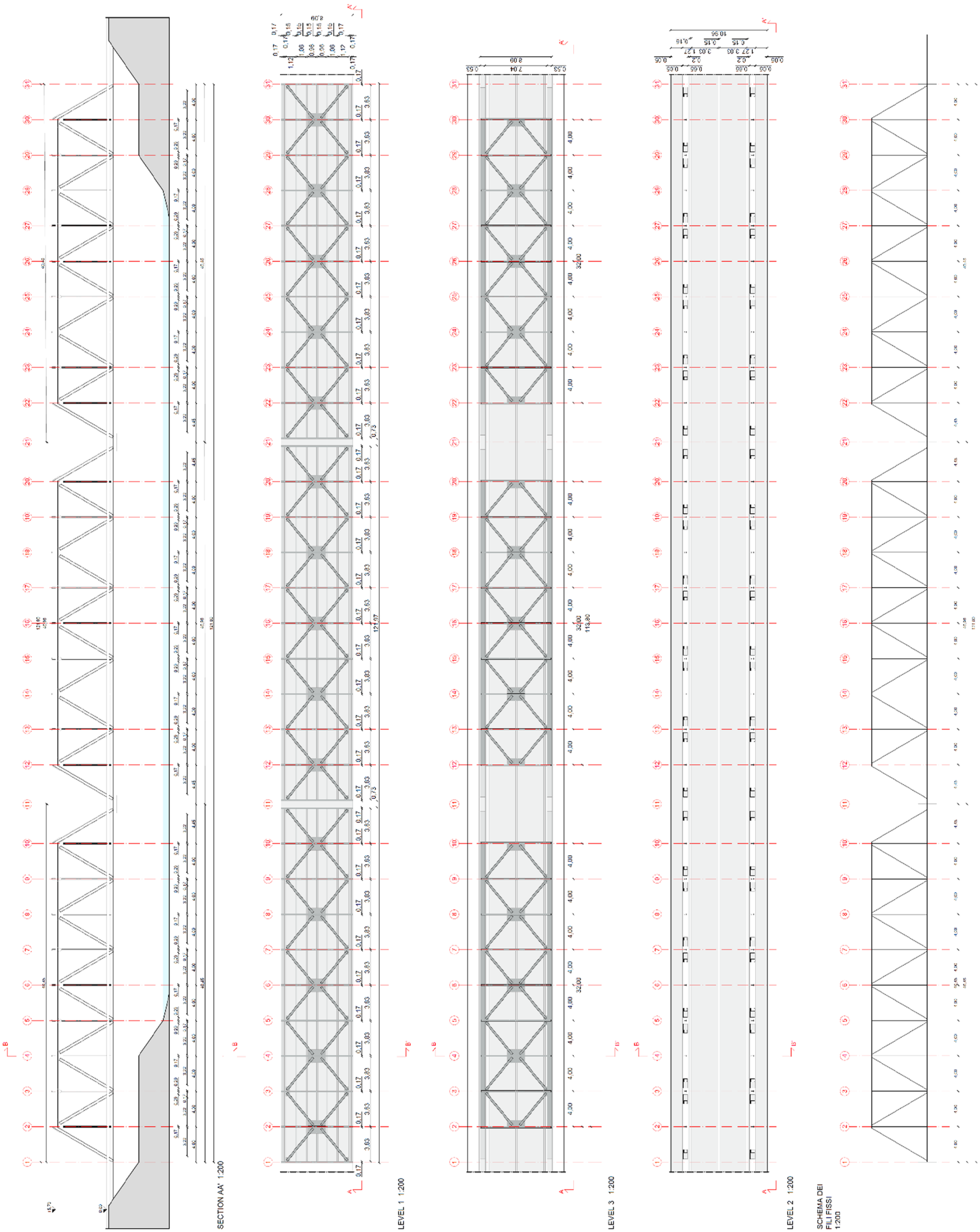


Figure 1. Bridge geometrical survey, main structure.

Figure 2. Lateral view of the bridge.

have been performed (e.g. lateral pedestrian passages, illumination plant, paintings), but they did not cover the entire structure. Deck member such as floor beams and lower chords have not been sandblasted and repainted and they are still in their original condition. The average depth of corrosion has been detected as approximately 1 mm, as no deeper corrosion has been found on structural members.

4.3. Structural modelling

The bridge structure was modelled using the finite element method (FEM) software Midas Civil (MIDAS, 2016), using only beam elements. Rigid links (rigid body) were used to represent eccentricities of the elements. Overall, the entire bridge model consists of about 2000 beam elements. A Young's modulus of 210,000 MPa (N/mm^2), Poisson's ratio of 0.3 and a material density value of 7850 kg/m^3 (weight density of 76.98 kN/m^3) were used for the analyses. All beam member sections were modelled as the as-built structure, as measured during the geometrical survey. The bridge is subjected to permanent loads (self-weight of steel elements and nonstructural elements weight) and to variable loads (temperature and traffic).

Firstly, the bridge was checked referring to the Italian Ministerial Decree 09/06/1945, n. 6018 (DM 6018, 1945) and to current Italian and European codes. The FEM model of one span has been calculated (Figures 4–8). The corroded 3-D frame girder model considered a cross-section reduction of 1 mm extended to the entire structure according to similar case studies (Brencich & Gambarotta, 2009; Pipinato et al., 2012), results have highlighted that the structural strength is not substantially affected by this grade of corrosion.

4.4. Structural calibration

Experimental data are needed whenever a theoretical model used for the assessment of a structure needs to be validated. For the case of existing bridges, either dynamic (Tobias et al., 1996; Calçada et al., 2002) and static load tests can be used because both provide synthetic information (natural modes and frequencies, displacements of relevant points) representing the overall response of the bridge; local measurements, such as strain gauges, are sometimes used for additional information. Two tests have been used for the structural calibration:

- SLD1: Historical static load tests are used to corroborate the theoretical model (Table 2); reference loads are those provided by the Italian Ministerial Decree (1945);
- SLD2: more recent static load tests performed in 2008 are used to corroborate the theoretical model further (Table 2); reference loads remain those provided by the Italian Ministerial Decree (2008).

4.5. Boundaries calibration

The boundary conditions of the bridge are presented in Figures 9–11: rollers (type A) and pinned (type B) supports are installed. According to a thermal FEM analysis performed (see Figures 12 and 13), the thermal expansion from 10 °C to 45 °C causes a maximum displacement of 3.6 mm. Considering that all the supports ensure a small translation of the floor beams, which has been measured to be up to 4 mm, the horizontal translation in the transverse direction is released. In the longitudinal direction, the displacement is opposed by a frictional force at the steel–concrete interface, directly proportional to the reaction force from the superstructure to the abutment:

$$F = \mu_s N \tag{1}$$

where F = frictional force, N = normal force (weight), μ_s is the friction ratio, assumed to be 0.6 (NC, 2012). The thermal expansion from 10 to 45 °C also generates maximum longitudinal forces of about 930 kN if the horizontal constraint in the longitudinal direction is fixed. As this force is higher than the frictional force considering structural and traffic loads, the translation in the longitudinal direction is free.

4.6. Loading conditions

The loads considered for the assessment of the bridge are: (i) dead loads of the bridge; (ii) thermal loads according to EN 1991-1-5 (2009); (iii) live loads according to two alternatives: (iiia) historical design code or (iiib) actual design code (Table 3). Concentrated loads have been positioned accordingly to the design code indication adopted. Live Loads exceeding those provided by the Italian Ministerial Decree (2008) are not admitted onto this bridge.

5. Strengthening alternatives

To achieve the possibility of extending the lifetime of these bridges, various strengthening strategies could be adopted.

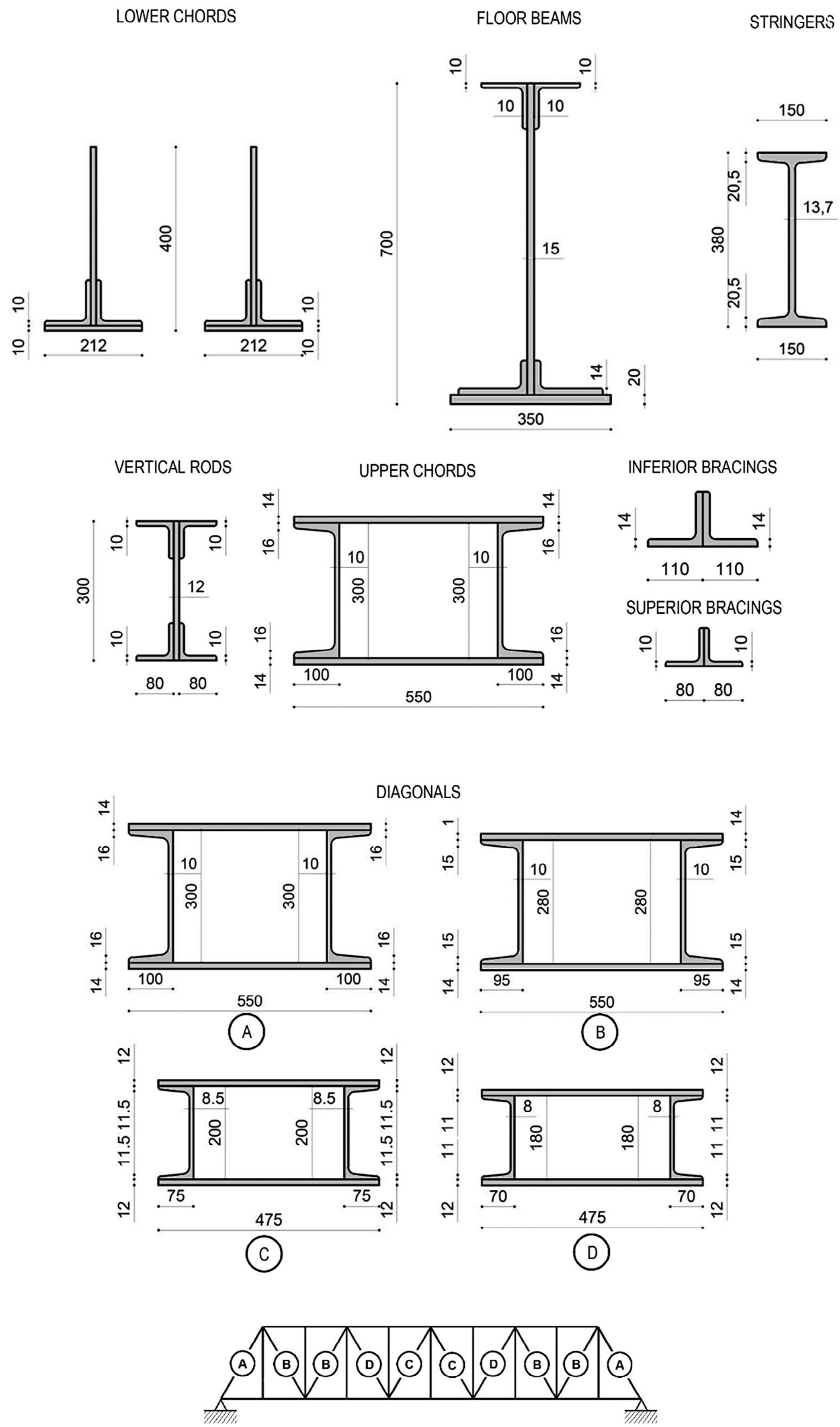


Figure 3. Bridge geometrical survey, structural details.

However, the first question to be solved is if the bridge could be used with a reduction of allowable live loads: in this case, most of the existing steel truss stock could be subjected to a structural analysis able to identify the allowable traffic loads, and then with usual maintenance interventions the bridge could be opened to traffic with a clear identification of the new bridge category. In

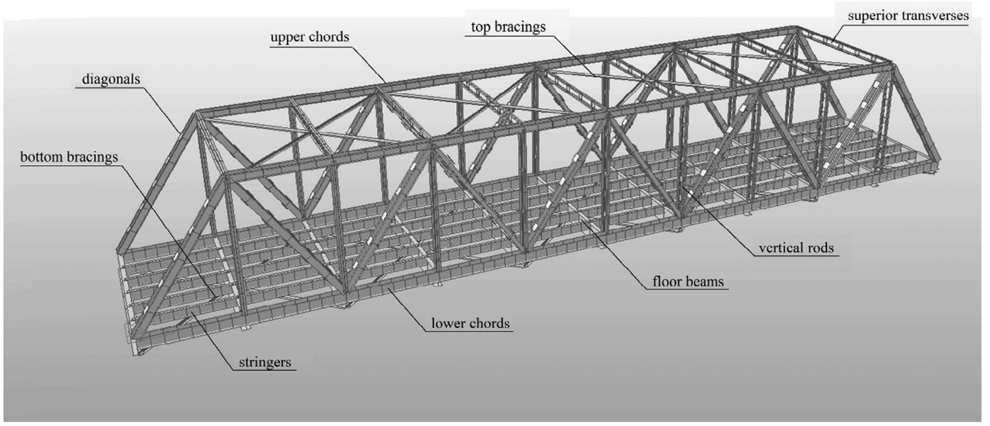


Figure 4. Adige Bridge, FEM model of one span.

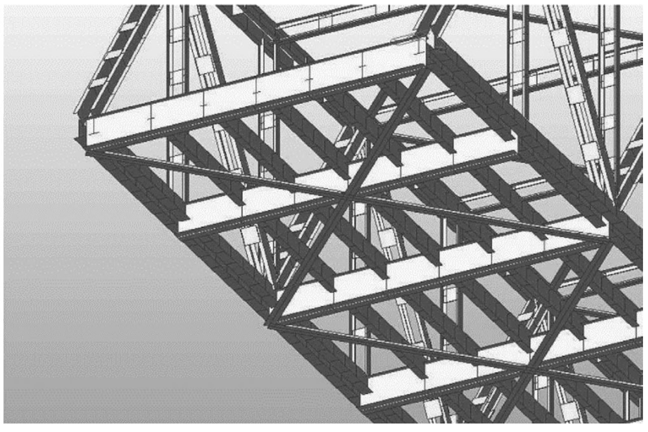


Figure 5. Detail of the deck (floor beams, stringers, and bracings).

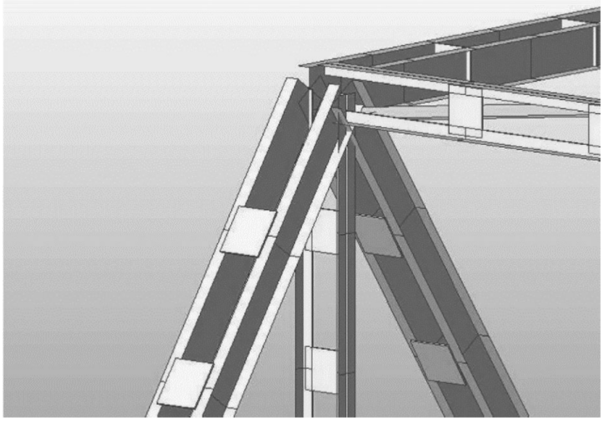


Figure 6. Detail of the diagonal–vertical truss node.

the case of Italy, the bridge category shifts from the 1st to the 2nd class (Italian Ministerial Decree, 2008):

- AD1 Load, 1st category: Load Model 1, EN 1991-2 (2003) (Table 3);

or

- AD2 Load, 2nd category: Load Model 1, EN 1991-2 (2003) taking a reduction of 20% for all loads of Lane number 1 (Table 3).

This second solution mainly applies for structures along secondary roads. For all other cases, or whenever the managing authority must ensure the Load Model 1 (EN 1991-2, 2003), strengthening strategies should be considered. In this study, strengthening alternatives chosen from the following solutions are considered:

(a) making composite an existing non-composite deck;
(b) building an orthotropic deck;
(c) building a new concrete deck, directly connected to the main trusses.

All these strategies are normally combined with steel-only interventions (including cover-plating, element replacement). The choice of the cover plating option (welded or bolted) represent a relevant issue for riveted bridges: in this particular case, considering the riveted built-up member geometry, and the weldability of the existing steel, welded cover-plates will be adopted; in conjunction with nodes, existing rivets will be replaced with high strength bolts, and cover-plates properly designed with bolting holes. Furthermore, the weldability of the base material of the whole bridge is assured by the original specification of the bridge components.

The described procedures are finalised to redistribute the live loads adequately onto the deck with a new or modified deck. As it can be noticed in the following discussions, the beneficial use of a rigid deck is often the best solution able to extend the bridge life adequately. In the present study, the reference strengthening solutions considered are (Table 4):

BR00: The existing bridge is calculated with the historical live load, *HS-LOAD*, without any strengthening intervention; even if these models are no longer representative for modern/future road traffic, this calculation is useful both for the structural calibration of the Fem model, and also to compare the efficiency of the strengthening solution proposed in the sequence.

BR01: The existing bridge is calculated with the actual code live load, AD1-LOAD, without any strengthening intervention.

BR02: The existing bridge is calculated with the actual code live load, AD2-LOAD, without any strengthening intervention.

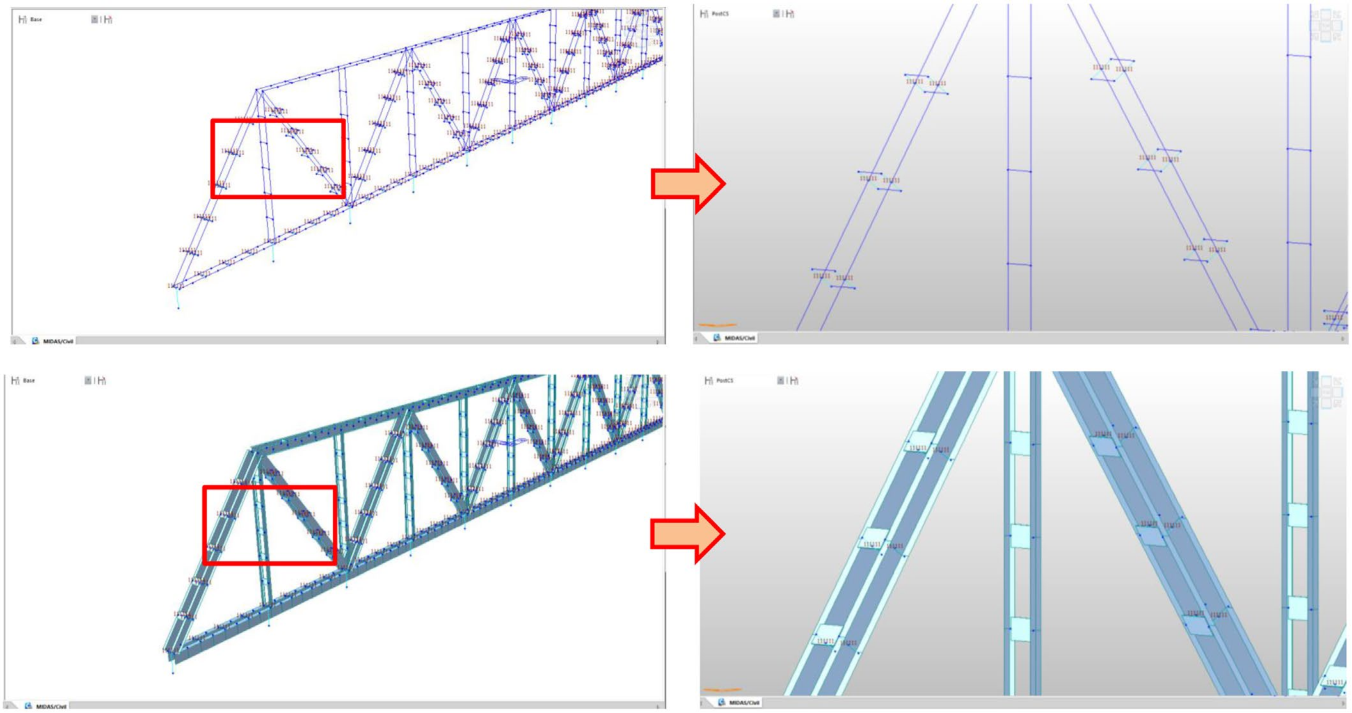


Figure 7. Detail of the diagonal and vertical truss element rigid-link and offset.

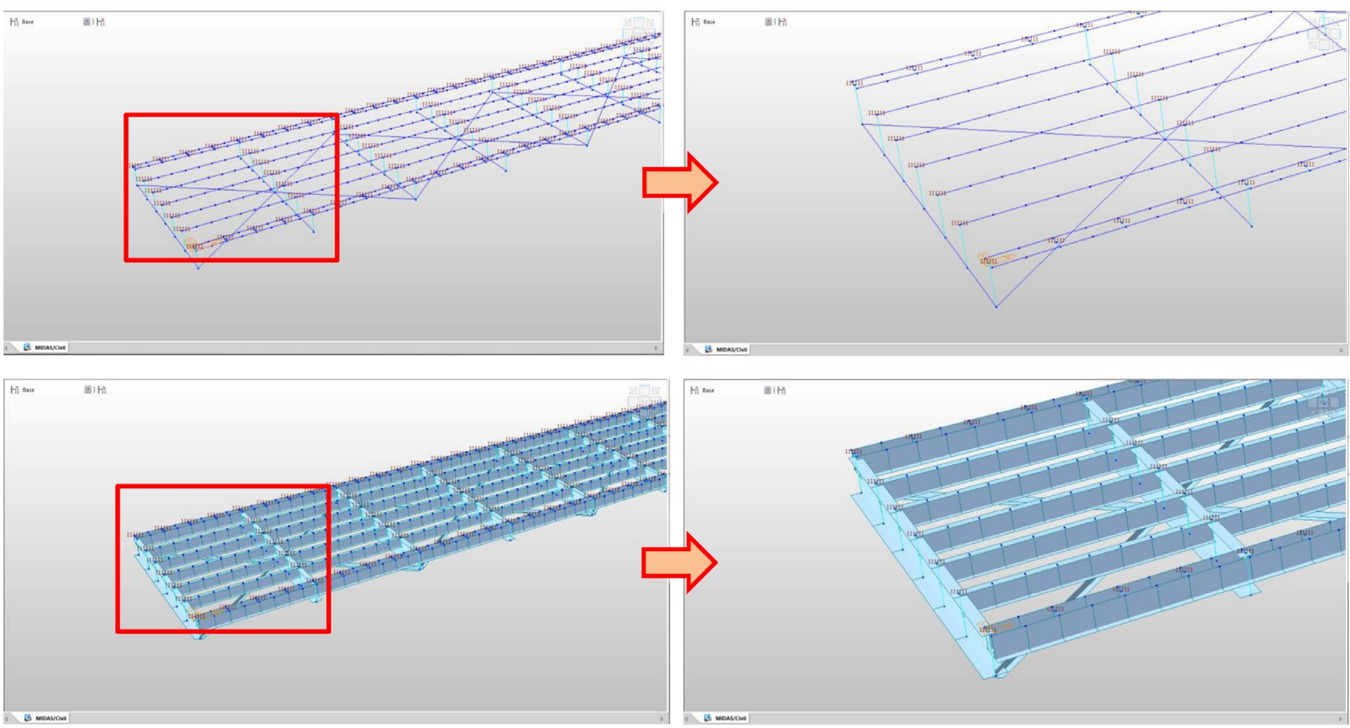


Figure 8. Detail of the deck beam offset.

Table 2. Theoretical vertical displacements of the nodes and measured values at the center of the bridge road: (a) topographic measurements; (b) FEM displacement.

Measure [mm]	Topographic SLD1	FEM SLD1	Topographic SLD2	FEM SLD2
1/3 of the span	−9.04	−9.17	−9.21	−9.54
Half span	−15.09	−14.47	−15.94	−15.24
2/3 of the span	−9.23	−9.17	−9.87	−9.54

RROOR1-2: The bridge is strengthened with the introduction of an orthotropic deck laying between stringers, with open ribs considering both the *AD1-Load* and *AD2-Load* (details are shown in Figure 14).

RROCR1-2: The bridge is strengthened with the introduction of an orthotropic deck laying between

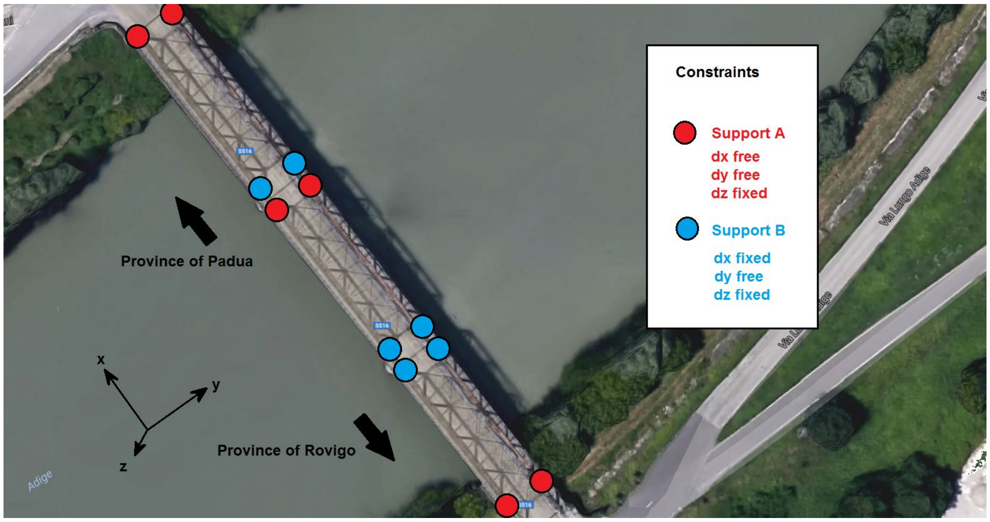


Figure 9. Bridge constraints.

Figure 10. Detail of support A and B (Padua side pier).

Figure 11. Detail of support A (Padua side pier).

stringers, with closed ribs considering both the *AD1-Load* and *AD2-Load* (details are shown in Figure 14).

RRA1-2: The bridge is strengthened with the introduction of a concrete deck considering both the *AD1-Load* and *AD2-Load* (details are shown in Figure 15); welded shear studs are introduced in the bridge to connect the new concrete deck with the stringers (three Ø 20 mm studs/m along all stringers, except for stringers along sections 1, 9–13, 19–23, 29–31 (see Figure 1, where six φ20 mm studs/m have been placed). A parametric analysis is performed varying both the concrete deck thickness (with a fixed strength of C40/50) among 10 cm (RRA1, 2–10), 15 cm (RRA1, 2–15), 20 cm (RRA1, 2–20), 25 cm (RRA1, 2–25), 30 cm (RRA1, 2–30) and the deck concrete strength (fixing the deck thickness at the lowest value of 10 cm) among C30/37 (RRA1, 2-R1), C35/45 (RRA1, 2-R2), C40/50 (RRA1, 2-R3), C45/55 (RRA1, 2-R4), C55/67 (RRA1, 2-R5).

RRA1-2-I: The bridge is strengthened with the introduction of a concrete deck considering both the *AD1-Load* and *AD2-Load* (details in Figure 15), fixing the concrete strength at C40/50 and the deck thickness at 100 mm; moreover, the steel-to-steel intervention described in Figure 16 are introduced adopting S355 new members.

RRB1-2-I: The bridge is strengthened with the introduction of a concrete deck considering both the *AD1-Load* and *AD2-Load* (details in Figure 17), introducing a UHPC concrete of C90/105 strength class and fixing the deck thickness at 50 mm; moreover, the steel-to-steel intervention described in Figure 18 are introduced adopting S355 new members.

RRC1-2-I: The bridge is strengthened with the introduction of a concrete deck considering both the *AD1-Load* and *AD2*-Load (details in Figure 19), introducing a UHPFRC concrete of C150/160 strength class and fixing the deck thickness at 30 mm; moreover, the steel-to-steel intervention described in Figure 20 are introduced adopting S355 new members.

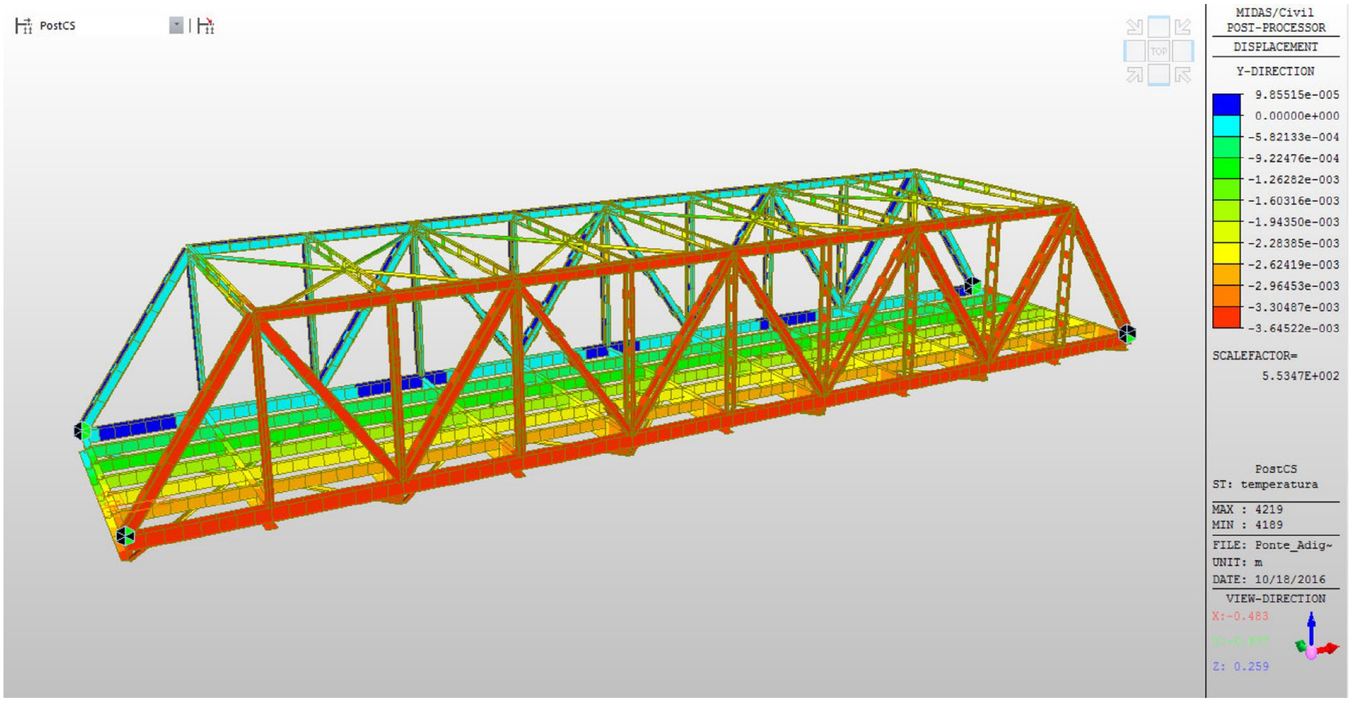


Figure 12. Boundaries calibration, thermal analysis: displacements releasing transversal movements (units in metre).

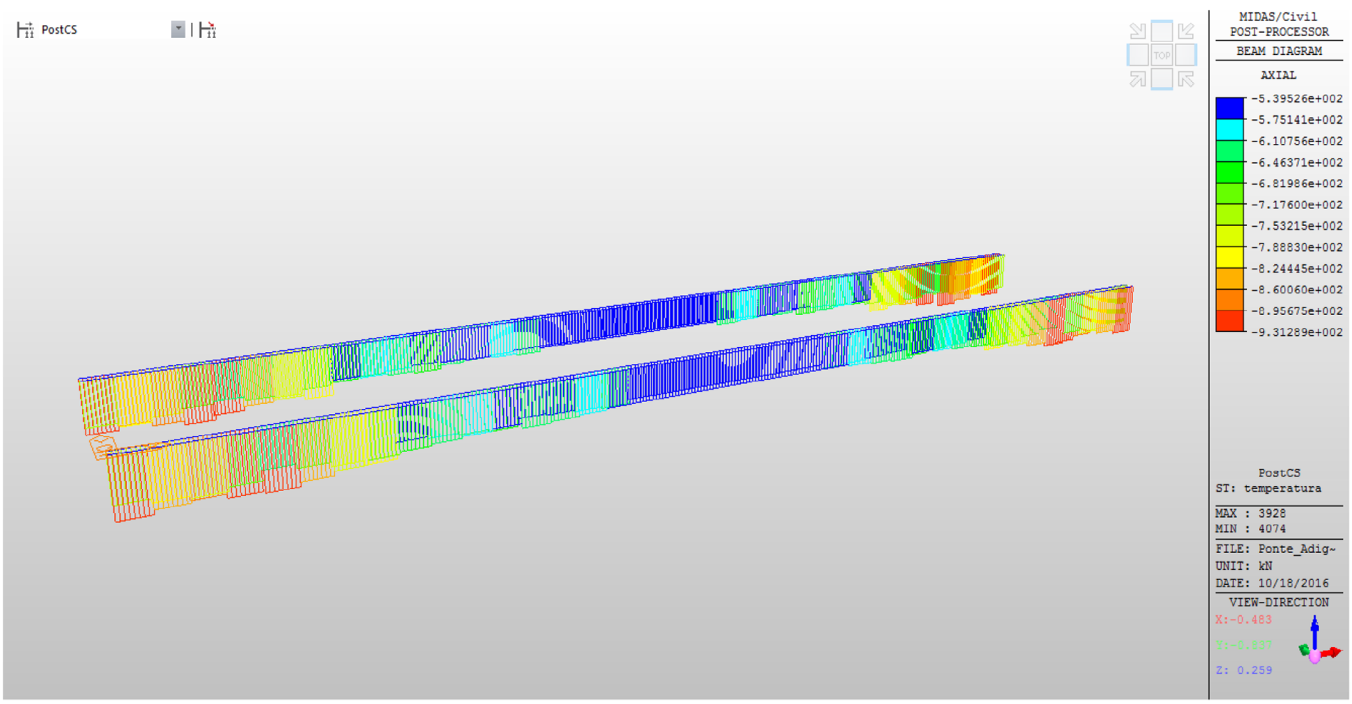


Figure 13. Boundaries calibration, thermal analysis: axial forces with unreleased transversal movements (units in kN).

For each alternative, Table 4 includes the following details: the strengthening code, the bridge load category (AD1 or AD2), the presence or not of a new steel–concrete composite deck, the presence or not of a new orthotropic deck, the presence or not of added steel members, synthetic details of the proposed strengthening method.

6. Verification procedure

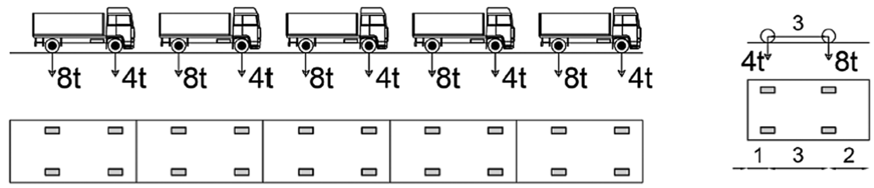
6.1 ULS and SLS general verification

The optimisation is calculated at the Ultimate Limit State (EN 1993-1-1, 2005) checking the safety factor of all members for all alternatives considered, according to the following specifications.

Table 3. Design load models adopted in the analysis.

Load type — **Load magnitude and disposition**

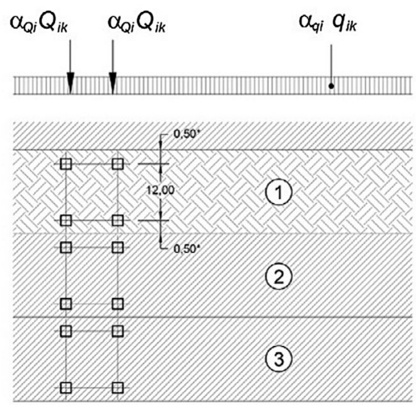
HS- LOAD: Historical design code : Italian Ministerial Decree 09/06/1945, n.6018



Indefinite double-axle trucks + additional load of 4kN/m² positioned along the whole deck.

AD-LOAD: Actual design code: Load Model 1 (EN 1991-2, 2003)

Location	TS	*UDL* system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{rk}) (kN/m²)
Lane 1	300	9
Lane 2	200	2,5
Lane 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5



Key
(1) Lane Nr.1: Q_{1k}= 300 kN; q_{1k}= 9 kN/m²
(2) Lane Nr.2: Q_{2k}= 200 kN; q_{2k}= 2,5 kN/m²
(3) Lane Nr.3: Q_{3k}= 100 kN; q_{3k}= 2,5 kN/m²
* For w_l= 3,00 m

The design value of the tension force N_{Ed} at each cross-section shall satisfy:

$$\frac{N_{\mathrm{Ed}}}{N_{t,\mathrm{Rd}}} \leq 1.0, \tag{2}$$

where:

$$N_{t,\mathrm{Rd}} \text{should be taken as the smaller of:} \begin{cases} N_{\mathrm{pl,Rd}} = \frac{A \cdot f_y}{\gamma_{M0}} \\ N_{u,\mathrm{Rd}} = \frac{0.9 \cdot A_{\mathrm{net}} \cdot f_u}{\gamma_{M2}} \end{cases},$$

where A_{net} is the net area of a cross-section.

The design value of the compression force N_{Ed} at each cross-section shall satisfy:

$$\frac{N_{\mathrm{Ed}}}{N_{c,\mathrm{Rd}}} \leq 1.0, \tag{3}$$

where:

$$\begin{cases} N_{c,\mathrm{Rd}} = \frac{A \cdot f_y}{\gamma_{M0}} \text{ for class 1, 2 or 3 cross-sections} \\ N_{c,\mathrm{Rd}} = \frac{A_{\mathrm{eff}} \cdot f_y}{\gamma_{M0}} \text{ for class 4 cross-sections} \end{cases}$$

where A_{eff} is the effective area of a cross-section.

The design value of the bending moment M_{Ed} at each cross-section shall satisfy:

$$\frac{M_{\mathrm{Ed}}}{M_{c,\mathrm{Rd}}} \leq 1.0, \tag{4}$$

$$\text{where:} \begin{cases} M_{c,\mathrm{Rd}} = M_{\mathrm{pl,Rd}} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} \text{ for class 1 or class 2 cross-sections} \\ M_{c,\mathrm{Rd}} = M_{\mathrm{pl,Rd}} = \frac{W_{el,min} \cdot f_y}{\gamma_{M0}} \text{ for class 3 cross-sections} \\ M_{c,\mathrm{Rd}} = M_{\mathrm{pl,Rd}} = \frac{W_{eff,min} \cdot f_y}{\gamma_{M0}} \text{ for class 4 cross-sections} \end{cases}$$

Table 4. Analysis summary: original bridge and retrofit solution analysis considered.

Bridge model code Ref.	Bridge loads	New steel- concrete composite deck	New ortho- tropic deck	Structural Steel intervention	Details
BR00	HS- LOAD				Original bridge
BR01	AD1 Load				Original bridge
BR02	AD2 Load				Original bridge
RROOR1	AD1 Load		X		Deck plate 20 mm, open rib tipe
RROOR2	AD2 Load		X		Deck plate 20 mm, open rib tipe
RROCR1	AD1 Load		X		Deck plate 20 mm, closed rib tipe
RROCR2	AD2 Load		X		Deck plate 20 mm, closed rib tipe
RRA1-10	AD1 Load	X			
RRA2-10	AD2 Load	X			
RRA1-15	AD1 Load	X			
RRA2-15	AD2 Load	X			
RRA1-20	AD1 Load	X			
RRA2-20	AD2 Load	X			
RRA1-25	AD1 Load	X			
RRA2-25	AD2 Load	X			
RRA1-30	AD1 Load	X			
RRA2-30	AD2 Load	X			
RRA1-R1	AD1 Load	X			
RRA2-R1	AD2 Load	X			
RRA1-R2	AD1 Load	X			
RRA2-R2	AD2 Load	X			
RRA1-R3	AD1 Load	X			
RRA2-R3	AD2 Load	X			
RRA1-R4	AD1 Load	X			
RRA2-R4	AD2 Load	X			
RRA1-R5	AD1 Load	X			
RRA2-R5	AD2 Load	X			
RRA1-R6	AD1 Load	X			
RRA2-R6	AD2 Load	X			
RRA1-I	AD1 Load	X		X	Using R.C. deck C40/50 100 mm thick
RRA2-I	AD2 Load	X		X	Using R.C. deck C40/50 100 mm thick
RRB1-I	AD1 Load	X		X	Using R.C. deck UHPC C90/105 50 mm thick
RRB2-I	AD2 Load	X		X	Using R.C. deck UHPC C90/105 50 mm thick
RRC1-I	AD1 Load	X		X	Using UHPFRC deck C150/160 30 mm thick
RRC2-I	AD2 Load	X		X	Using UHPFRC deck C150/160 30 mm thick

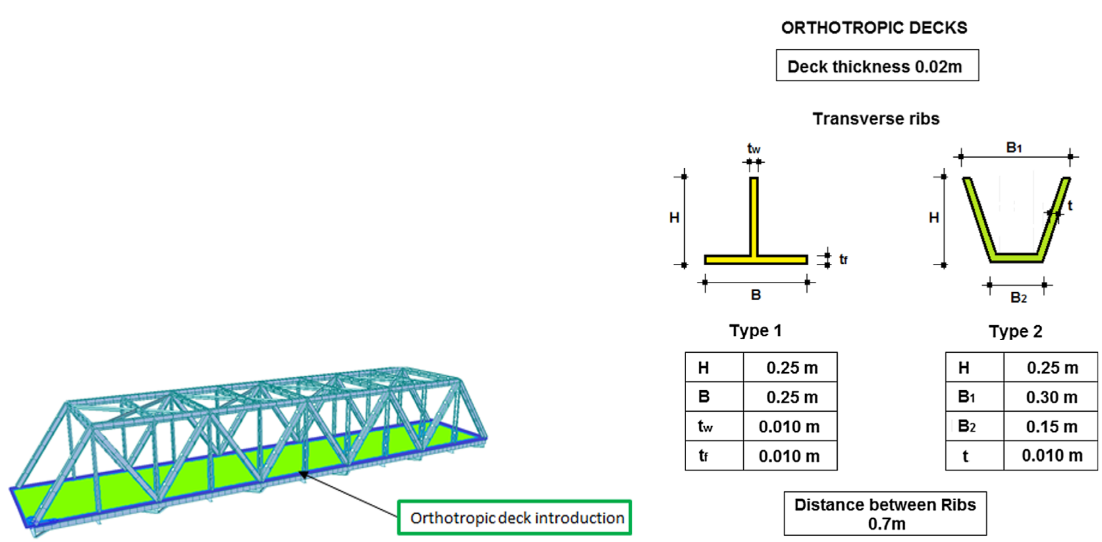


Figure 14. Orthotropic deck with open (RROOR1-2) or closed (RROCR1-2) stiffeners (ribs).

For class 1, class 2 or class 3 cross-sections subjected to the combination of N_{Ed}, $M_{y,Ed}$, $M_{z,Ed}$, this method may be applied using the following criteria:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{Rd}}} + \frac{M_{y,\mathrm{Ed}}}{M_{y,\mathrm{Rd}}} + \frac{M_{z,\mathrm{Ed}}}{M_{z,\mathrm{Rd}}} \leq 1.0, \tag{5}$$

where N_{Rd}, $M_{y,Rd}$, $M_{z,Rd}$ are the design values of the resistance. For biaxial bending the following criterion may be used:

$$\left[\frac{M_{y,\mathrm{Ed}}}{M_{y,\mathrm{Rd}}}\right]^{\alpha} + \left[\frac{M_{z,\mathrm{Ed}}}{M_{z,\mathrm{Rd}}}\right]^{\beta} \leq 1.0, \tag{6}$$

in which α and β are constants, which have been conservatively be taken as unity, according to EN 1993-1-1 (2005), par. 6.2.9.1.

All members have been grouped into the following subsets (cf. Figures 3 and 6): lower chords, struts, stringers, bottom bracing, floor beams, diagonals_A, diagonals_B, diagonals_C, diagonals_D, upper chords, superior transverses, top bracings. The analysis results are illustrated in Table 5, reporting for each alternative the maximum ratios E_d/R_d defined as the minimum safety factor of all ultimate limit state (ULS) checks mentioned herein (MIDAS, 2016); ratios less than 1 imply that all the member strength verifications are verified. In the composite section case, for example, when stringers are modified to composite sections in the strengthening solution analysed, the symbol 'V' implies that the ultimate limit state (ULS) verification of the composite section is satisfied. Table 5 does not report results of the upper category (AD1 Load) if the minor (AD2 Load) fails the verification checks. Moreover, serviceability limit state (SLS) verifications are performed.

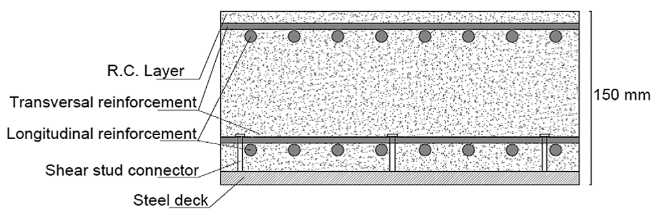


Figure 15. Composite deck retrofit alternative RRA1,2 (the deck width is variable).

6.2 Fatigue verification

Considering that one of the relevant structural aspects of existing steel bridge strength must face the fatigue damage issue, a description of the adopted procedure for fatigue assessment

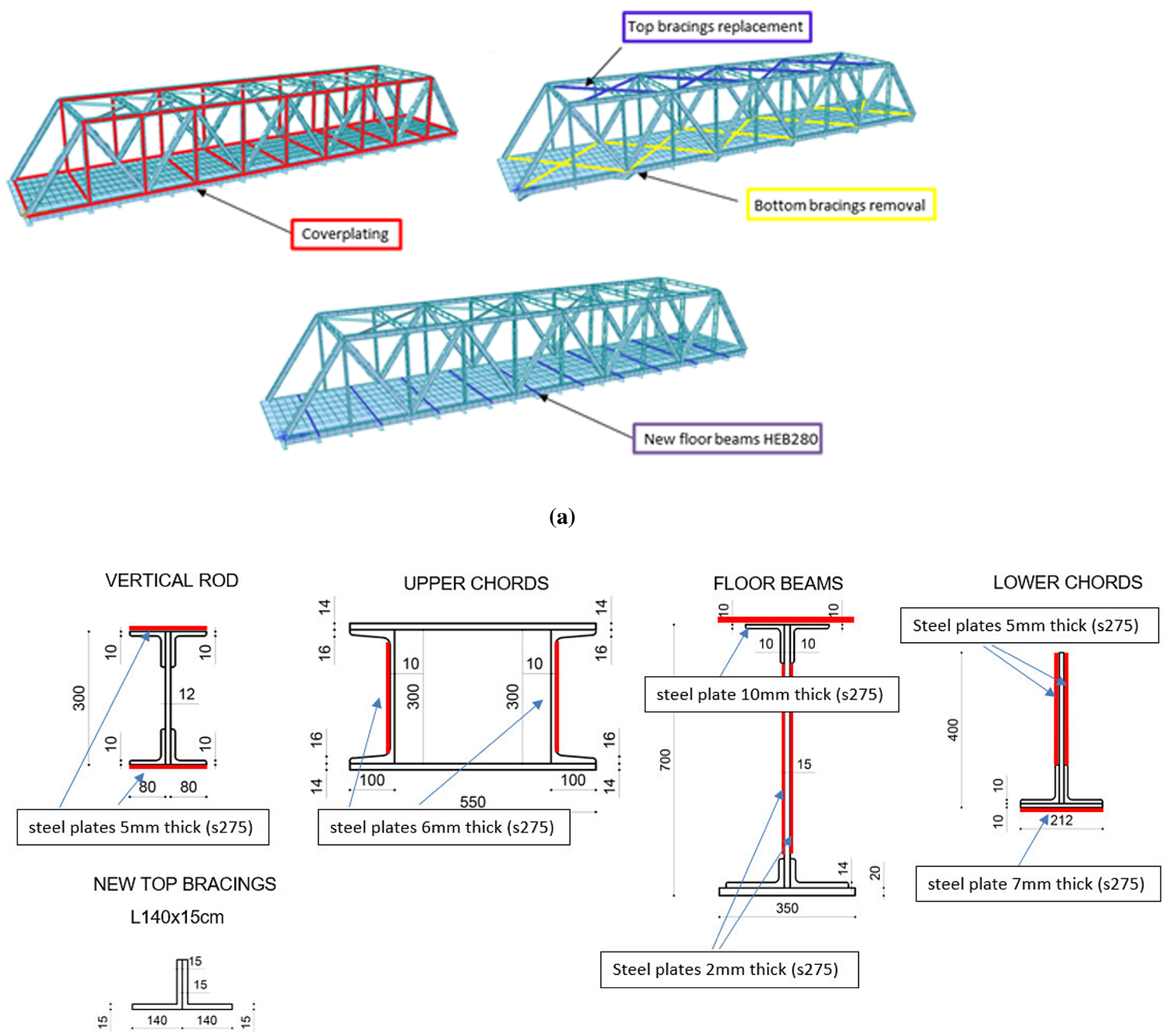


Figure 16. Steel to steel retrofit intervention for the retrofit alternative RRA1-2-I: general description of the interventions (a); cover-plating and replaced new members (b).

is herein presented according to the EN 1993-1-9 (2005). The verification implies the calculation of coded parameters as: $\Delta\sigma$ (stress range for direct stress), $\Delta\sigma_C$ (reference value of the fatigue strength at NC = 2 million cycles), λi (damage equivalent factors), $\Delta\sigma_{E,2}$ (equivalent constant amplitude stress range). Even if a general verification of the whole structure with Load Model 1

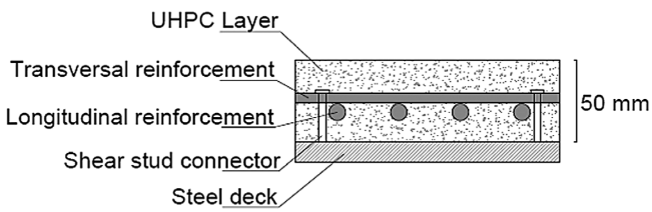
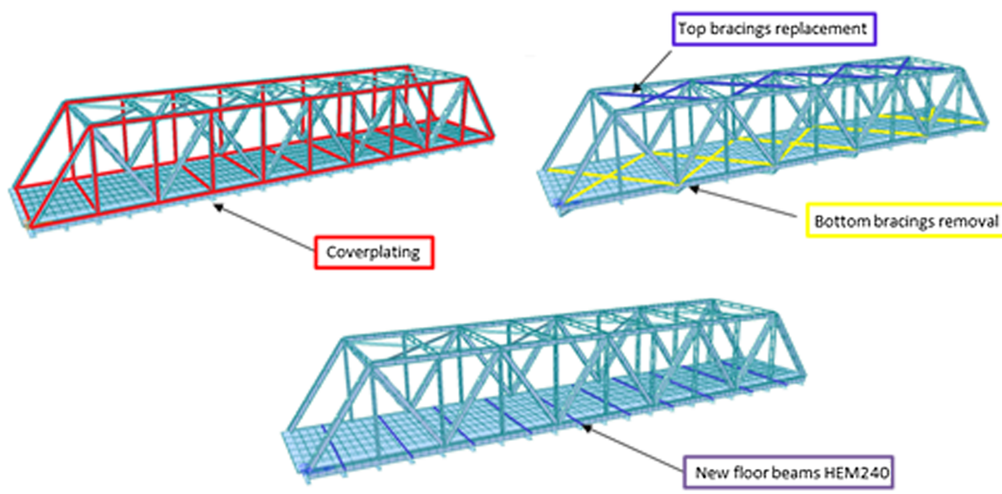


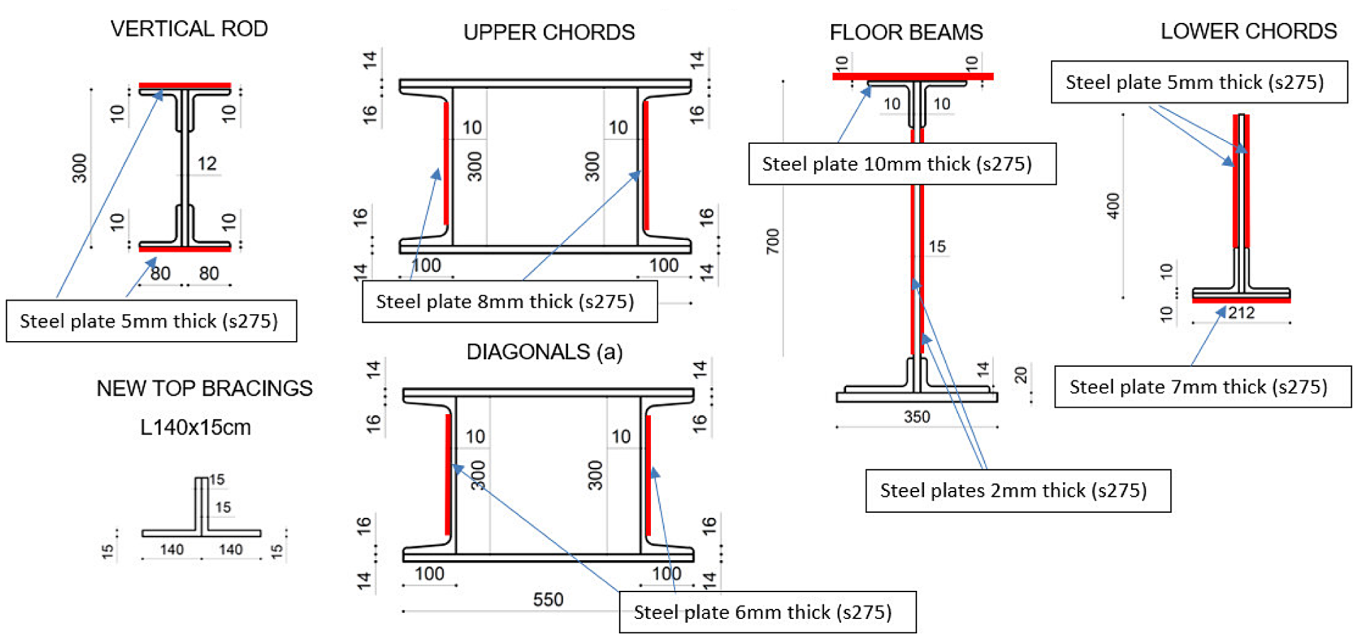
Figure 17. Deck retrofit alternative RRB1.

has been performed for all solutions, this could be could be very conservative; in fact with this procedure some structural component could be not verified (e.g. deck tranverse). Fatigue Load Model 1 has the configuration of the characteristic Load Model 1 defined in EN 1991-2 (2003), with the values of the axle loads equal to 0.7Qik and the values of the uniformly distributed loads equal to 0.3q_{ik} and (unless otherwise specified) 0.3q_{rk}.

The load values for Fatigue Load Model 1 (FLM 1) are similar to those defined for the Frequent Load Model, however adopting the Frequent Load Model without adjustment would have been excessively conservative by comparison with the other models, especially for large loaded areas. For this reason, fatigue verification have been performed with the Fatigue Load Model 3 defined in EN 1991-2 (2003) and the equivalent damage procedure: this model consists of four axles, each of them having two identical



(a)



(b)

Figure 18. Steel to steel retrofit intervention for the retrofit alternative RRB1-2-I: general description of the interventions (a); cover-plating and replaced new members (b).

wheels, the weight of each axle is equal to 120 kN, and the contact surface of each wheel is a square of side 0.40 m (see par. 4.6.4 of EN 1991-2 (2003)). The damage equivalence factor λ for road bridges up to 80 m span should be obtained from:

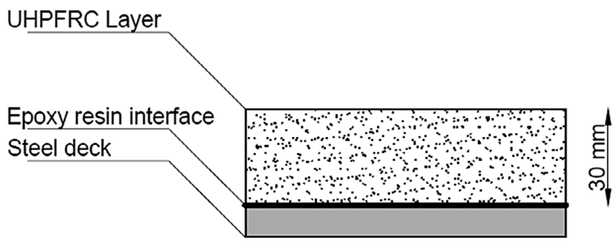


Figure 19. Deck retrofit alternative RRC1.

$$\lambda = \lambda 1 \times \lambda 2 \times \lambda 3 \times \lambda 4 \text{ but } \lambda \leq \lambda_{max} \tag{7}$$

where $\lambda 1$ is the factor for the damage effect of traffic and depends on the length of the critical influence line or area; $\lambda 2$ is the factor for the traffic volume; $\lambda 3$ is the factor for the design life of the bridge; $\lambda 4$ is the factor for the traffic on other lanes; λmax is the maximum λ-value taking account of the fatigue limit.

The fatigue assessment should be carried out as follows:

$$\gamma_{Ff} \Delta\sigma_{E2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \tag{8}$$

Fatigue verification have not been performed for those alternative that do not fulfil the ULS; however, it should be noted

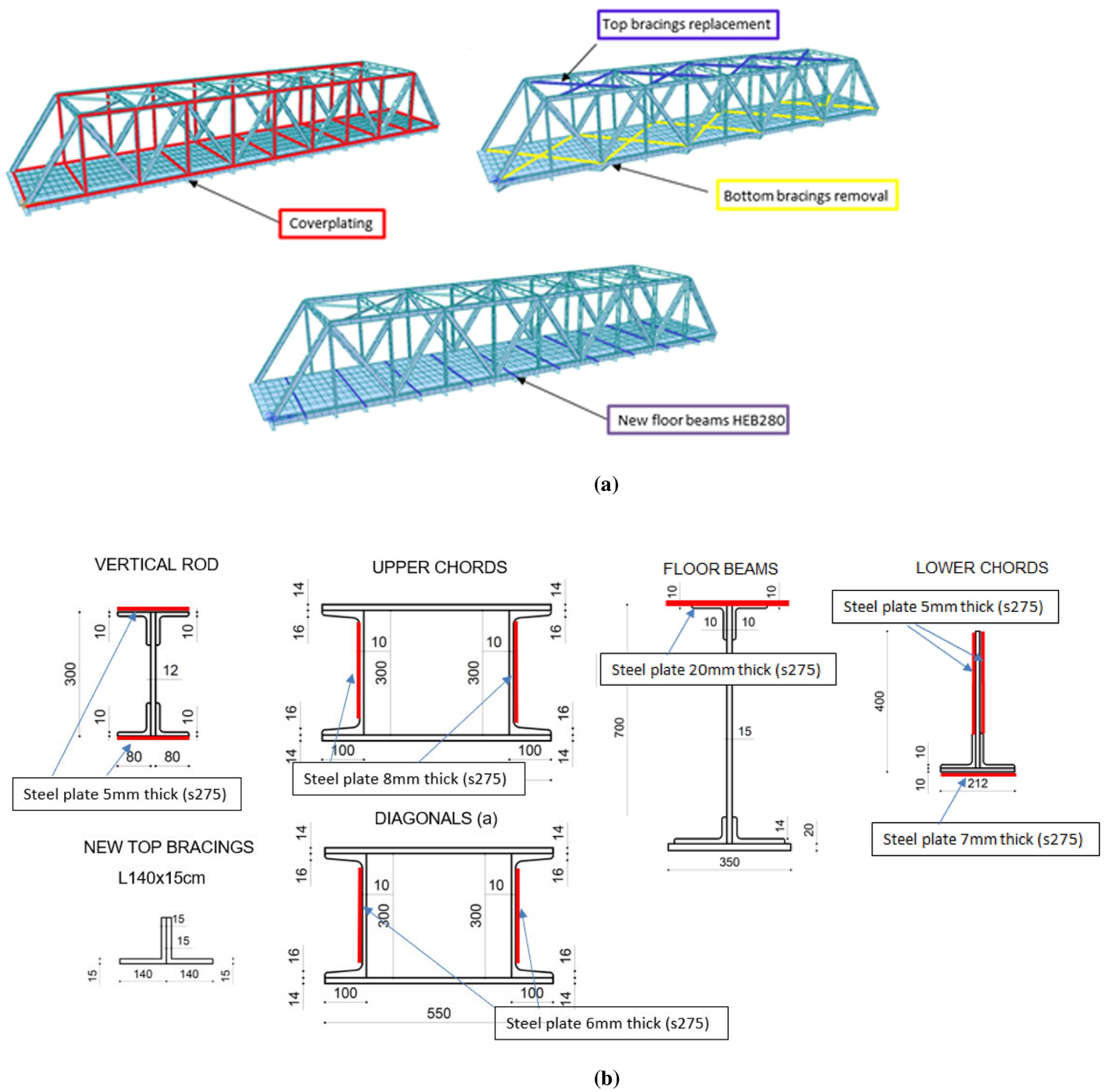


Figure 20. Steel to steel retrofit intervention for the retrofit alternative RRC1-2-I): general description of the interventions (a); cover-plating and replaced new members (b).

Table 5. Retrofit solution alternatives considered and safety factors associated.

Retrofit code Ref.	Lower Chords	Vertical Rods	Stringers	Bottom Bracings	Floor Beams	Diago-nals_A	Diago-nals_B	Diago-nals_C	Diago-nals_D	Upper Chords	Superior Trans-verses	Top Brac-ings
BR00	0.572	0.230	0.096	0.144	0.315	0.366	0.319	0.195	0.237	0.335	0.132	0.642
BR01	2.114	inf.	inf.	0.715	5.261	5.164	3.015	1.402	3.326	1.020	0.360	1.555
BR02	1.865	inf.	6.806	0.629	4.482	1.767	1.333	1.208	1.221	0.895	0.315	10.787
RROOR2	4.115	2.135	4.319	1.430	1.996	1.249	1.015	1.001	0.873	1.015	0.353	1.551
RROCR2	4.055	1.960	3.938	1.524	1.967	1.241	1.006	0.993	0.864	1.006	0.350	1.537
RRA2-10	1.653	1.018	V	0.539	1.907	1.105	0.969	0.831	0.832	1.031	0.322	2.510
RRA2-15	1.727	1.086	V	0.522	1.708	1.183	1.051	0.891	0.893	1.104	0.338	4.140
RRA2-20	2.003	1.137	V	0.535	1.589	1.247	1.119	0.936	0.944	9.654	0.352	5.016
RRA2-25	2.282	1.178	V	0.559	1.644	1.305	1.181	0.972	0.986	inf.	0.366	3.111
RRA2-30	2.553	1.213	V	0.640	1.646	1.362	1.237	1.000	1.023	inf.	0.379	3.303
RRA2-R1	1.707	1.081	V	0.506	1.724	1.181	1.047	0.886	0.892	1.102	0.337	3.522
RRA2-R2	1.718	1.084	V	0.515	1.716	1.182	1.049	0.889	0.892	1.103	0.337	3.828
RRA2-R3	1.727	1.086	V	0.522	1.708	1.183	1.051	0.891	0.893	1.104	0.338	4.140
RRA2-R4	1.736	1.088	V	0.528	1.701	1.184	1.052	0.893	0.894	1.105	0.338	4.465
RRA2-R5	1.747	1.089	V	0.533	1.694	1.184	1.053	0.895	0.894	1.106	0.338	4.809
RRA2-R6	1.756	1.091	V	0.538	1.688	1.185	1.054	0.896	0.895	1.107	0.338	5.187
RRA1-I	1.000	0.930	V	0.951	0.722	0.959	0.929	0.820	0.824	0.984	0.303	0.168
RRB1-I	1.000	0.900	V	0.943	0.856	0.898	0.865	0.796	0.767	0.906	0.307	0.168
RRC2-I	1.346	0.814	V	1.485	1.046	0.893	0.830	0.720	0.793	0.903	0.295	0.167

Table 6. Fatigue verification according to EN 1991-2 (2003).

Fatigue load Model according to EN 1993-1-9 (2005)	Structural compo-nent	$\Delta\sigma$	$\lambda 1$	$\lambda 2$	$\lambda 3$	$\lambda 4$	λ	$\Delta\sigma_c$	Safe life procedure		Detail cat.	$\Delta\sigma_{E,2}$	
		[MPa]	[-]	[-]	[-]	[-]	[-]	[MPa]			[MPa]	[MPa]	
L.M. 3	Lower chords	28.8	2.25	0.362	0.871	1	0.709	20.4	High consequence	1.35	90	66.7	Verified
L.M. 3	Current floor beams	47.8	2.55	0.362	0.871	1	0.804	38.4	Low consequence	1.15	90	78.3	Verified
L.M. 3	HEB280 floor beams	75.3	2.55	0.362	0.871	1	0.804	68.5	Low consequence	1.15	90	78.3	Verified
L.M. 3	Diagonals	44.5	2.55	0.362	0.871	1	0.804	38.8	High consequence	1.35	90	66.7	Verified
L.M. 3	Vertical rods	36.7	2.55	0.362	0.871	1	0.804	35.8	High consequence	1.35	90	66.7	Verified
L.M. 3	Stringers	33.5	2.55	0.362	0.871	1	0.804	26.9	High consequence	1.35	120	88.8	Verified
L.M. 3	Lower chords weld/ cover-plates	40.8	2.25	0.362	0.871	1	0.709	28.9	High consequence	1.35	56	41.5	Verified
L.M. 3	Diagonals weld/cov-er-plates	36.4	2.55	0.362	0.871	1	0.804	29.3	High consequence	1.35	56	41.5	Verified
L.M. 3	Vertical rods weld/ cover-plates	39.3	2.55	0.362	0.871	1	0.804	31.6	High consequence	1.35	56	41.5	Verified

that for those cases in which the orthotropic steel deck solution is adopted in the retrofit alternative, the new fatigue-sensitive details must be assessed according to its fatigue strength category on the basis of 'Table 8.8: Orthotropic decks – closed stringers' and 'Table 8.9: Orthotropic decks – open stringers' (EN 1993-1-9, 2005). Moreover, the mentioned analyses are important to assess the fatigue performance of this specific solution, adopting both FLM 1 and 3; this last check (with FLM3) is of fundamental importance for orthotropic deck in order to understand concentrated loads fatigue effects due to the vehicle wheels instead of uniformly distributed loads as in the FLM1, and could reveal fatigue-sensitive regions for the orthotropic steel deck alternative. Not only the principle structure of the bridge have been checked for fatigue, but also cover-plating intervention have been analysed as it introduces newly fatigue-sensitive details in the bridge steel truss members. As cover plating is assumed to be built on the basis of Details 7) of Table 8.1 (EN 1993-1-9, 2005) the fatigue strength category is represented by $\Delta\sigma_C = 56$ MPa.

A conservative hypothesis concerning the adoption of the UHPC deck, which consists in the reduction of the deformation modulus of the ultra-high performance concrete of 50%, the worst found in literature. This decision is supported by recent investigations: e.g. Xu et al. (2017) conducted the fatigue test of the reactive powder concrete, which indicated a decrease of 50% in the original value of the elastic modulus before reaching the fatigue damage. Similar results are reported in Makita and Bruwhiler (2015) which investigated the damage models for UHPFRC and R-UHPFRC tensile fatigue behaviour finding a decrease of 30% in the original value of the elastic modulus before reaching the fatigue damage.

7. Discussion of the results

As can be inferred from the observation of the various structural analyses performed, only two alternatives satisfy the verification, RRA1-I and RRB1-I:

- This solution has been introduced because the previous parametric analysis (RRA class) has shown that introducing new orthotropic steel decks or a composite concrete deck improves the overall structural behaviour of the bridge compared with the original situation, even if ULS (ultimate limit state) checks were still not verified. In this case, the bridge has shown that the good capacity to redistribute

the live load demonstrated by the improved deck is not in line with the understrength capacities of other members (struts, upper chords, floor beams and lower chords). To improve the deck redistribution, new HEB280 floor beams are introduced, top bracing is replaced with bigger sections and the bracing geometry is changed (K-bracings are used despite the original X-bracings), while bottom bracing is removed because their original redistribution and stiffening action of the original only-steel deck is now performed by the new composite deck; serviceability limit state (SLS) verification performed has highlighted reduced displacements (max 3.02 cm) and compatible with the use of the bridge. Fatigue checks have been performed according to EN 1993-1-9 (2005) and have highlighted the fulfilled verification of the structure.
- This strengthening solution is conceived to lighten the weight of the slab while maintaining its stiffness. As shown before, a high concrete class deck ensures high performances with lower weight. The concrete deck is 5 cm thick (concrete C90/105) and new S355 steel HEB280 floor beams are introduced. Elements sections are reinforced, while the same interventions adopted in RRA1-I are conceived for the top and bottom bracing. Serviceability limit state (SLS) verification performed has highlighted reduced displacements (max 3.12 cm) compatible with the use of the bridge; fatigue checks have been performed according to EN 1993-1-9 (2005) and have highlighted the fulfilled verification of the whole structure adopting the Fatigue load Model 3 (EN 1991-2 (2003), Table 6). For comparison, verification with the Fatigue Load Model 1 are fulfilled for the whole structure except for the HEB 280 transverse that should be substituted with a HEM260 to fulfil the verification. A further ultimate limit state and serviceability limit state check of the bridge has then been performed.

Findings of the study are relevant and innovative due to:

- common procedures of intervention imply the use of new concrete deck resting on the existing steel structure with thickness of 30 cm (Simon-Talero & Merino, 2006; Brozzetti, 2000; Chung & Sotelino, 2006), or more
- the paper highlighted that the strengthening of steel trusses could be done by the adoption of high strength concrete with a thicker slab (50–100 mm), thus reducing the self-weight of the bridge; and this then becomes an extra live load allowance, able to cope with live loads in line with those requested by the current codes and standards. Actually, the deck thickness reduction using high strength material should be the correct way to pursue innovative solution for strengthening existing bridges: for this reason, further research is needed for the RRC2-I alternative. Recent research dealing with the fatigue behaviour of UHPFRC and R-UHPFRC has been investigated with the purpose of using this material for the fatigue strengthening of existing bridges (Lamine et al., 2013; Toutlemonde et al., 2007). Furthermore, other studies confirm that a very thick UHPC layer (50 mm) connected with short welded shear connectors could enhance the fatigue performance of an orthotropic existing deck (Zhang et al., 2016).

8. Conclusions

This paper proposes a novel approach for the strengthening of existing steel truss bridges, to extend their life with optimised interventions. Based on the current investigations, the following conclusions can be drawn:

(a) The existing bridge can carry the historical loads HS-LOAD (DM 6018, 1945); without any intervention.
(b) The existing bridge is not able to carry the actual loads AD-LOAD (EN 1993-2, 2006), without any intervention; also, a reduced live load (AD2 Load, second category bridge) is not able to be carried by the existing bridge.
c) Detailed parametric studies were performed for the strengthening alternatives to gain deeper insight into the structural behaviour of the bridge and of all structural members. A large number of structural alternatives have been considered, based onto the deck strengthening of existing steel truss bridge introducing orthotropic deck or composite deck.
d) The analyses show that the final optimised strengthening solution represented by a composite UHPC deck resting on the existing steel structure has an excellent structural performance compared with other alternatives via FEM and parametric analysis; this is also in line with recent research with similar deck strengthening solution calibrated with FEM and real scale testing Zhang et al. (2016).

Even if these results can suggest a precise structural strengthening alternative, further works are ongoing in the same research project, especially for the investigation of:

- the fatigue behaviour of the bridge encompassing a different range of traffic growth (both number of vehicles and weight of vehicles) than those provided on the official standard 1993-1-9 (2005);
- the influence of changing the bottom and top bracing system considering K- or X-bracings, to optimise the steel-to-steel intervention, to reduce to the minimum the increase of load due to the strengthening solution.

Disclosure statement

No potential conflict of interest was reported by the author.

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