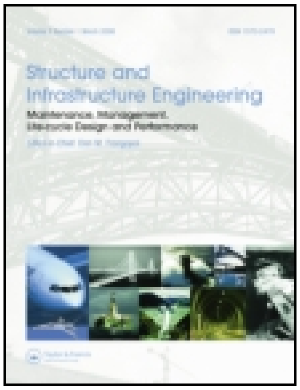

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Assessment procedure and rehabilitation criteria for the riveted railway Adige Bridge

A. Pipinato [a] , C. Pellegrino [a] & C. Modena [a]
[a] Department of Structural and Transportation Engineering, University of Padova, Italy

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Assessment procedure and rehabilitation criteria for the riveted railway Adige Bridge

A. Pipinato*, C. Pellegrino and C. Modena

Department of Structural and Transportation Engineering, University of Padova, Italy

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Traffic patterns increasing and the degradation of existing railway steel bridges has lead to the need of an assessment of their remaining fatigue life, and deciding whether to retrofit or supply structure replacement. This paper deals with the structural assessment of an actual case study, the Adige Bridge, which connects the Rovigo and Padua provinces in northern Italy. The bridge has been in service since 1886 and the overall length is about 161m through three spans. As a reference the innovative procedures to estimate the remaining fatigue life of bridges outlined in the JRC-ECCS document *Assessment of Existing Steel Structures: Recommendations for Estimation of Remaining Fatigue Life*, has been applied for the case study, together with other codes and technical instructions. Stress data, obtained by a 3D finite element model, were used to estimate the remaining fatigue life. Assessment results, obtained by considering different traffic estimations, point out that most of the identified critical details have an infinite remaining safe life, but at the same time some members appear critical. Appropriate retrofitting criteria are proposed to support the designer in common damage situations.

Keywords: railway; steel bridge; rivets; high-cycle fatigue; rehabilitation

Introduction

Bridges are a strategic part of ancient rail networks and in some cases they are at the limit of their traffic capacity. In particular, riveted constructions were widely used in railway bridges during the second half of the nineteenth century up to the middle of the twentieth. Most of these wrought-iron or older steel bridges are still in use around Europe and particularly in Italy. Sixty per cent of Italian railway steel bridges are about 100 years old as they were built between 1900 and 1920. They were not designed explicitly against fatigue but have sustained continuously increasing loads and appear able to cope with current loading demands. Nonetheless, the lack of a comprehensive fatigue assessment methodology for riveted bridges, combined with the uncertainty associated with the fatigue phenomenon and the discovery of hidden cracks, has led to research initiatives investigating the fatigue behaviour of riveted railway bridges. Moreover, the ASCE Committee on Fatigue and Fracture Reliability (1982) reported that 80–90% of failures in steel structures are related to fatigue and fracture. Several factors play an important role as documented by several researches, such as Bruhwiler *et al*. (1990), Kulak (1992), Akesson and Edlund (1994), Di Battista *et al*. (1997), Bursi *et al*. (2002), Matar and Greiner (2006), Boulent (2008), Pipinato (2008), Albrecht and Lenwary (2008, 2009), Pipinato *et al*. (2009, 2010). With regard to loadings, the dead load vs. live load ratio is usually about 15–20%, thus implying that railway bridges are subjected to large variations of live load-induced stresses. Moreover, geometric imperfections, such as the inclination and/or deflection of structural elements, entail secondary stresses that are not usually taken into account in fatigue assessments. Vibrations, transverse horizontal forces, internal constraints, localised and diffused defects as corrosion damages, are concurring causes of fatigue damage (Byers *et al*. 1997). Moreover, the use of different riveting techniques may entail variable clamping force levels and load carrying capacities in members and joints. Also the presence of several joints, detail sizes and different materials in the same bridge induce different types of fatigue resistance. As a matter of fact, the most critical fatigue details in riveted railway bridges are floor-beam tension hanger and stringer-to-floor connections for medium and long span girder bridges (Al-Emrani 2000, 2005) and short shear diaphragm for small span bridges (Pipinato *et al*. 2009): these represent hot governing spot details in which alternating stress decrease the fatigue strength of the whole structure. In situ and laboratory procedures, which are often not mentioned in code assessment procedures, increase the knowledge of material properties where they are not known (Ermopoulos 2006,

*Corresponding author. Email: alessio.pipinato@unipd.it

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Farhey *et al*. 1997) and their effect on fatigue life evaluation.

In this paper, a 64-year-old steel bridge over the Adige Bridge near Rovigo in the north eastern part of Italy, a typical railway bridge of the mid-nineteenth century, is studied according to a step-level assessment procedure.

First the bridge is geometrically described and a material investigation is carried out. Then, a FEM model is used to find out critical details. Finally, critical stress data are used in order to perform the remaining fatigue life assessment and to give repair indications.

The aim of this study consists in the development of a simplified end-user step-level procedure for fatigue assessment for railway bridges and application to a significant case study. Appropriate retrofitting criteria are also proposed to support the designer in common damage situations. This aim is suggested by the diffused necessity of the authorities managing railway lines, since a number of steel bridges with a static scheme similar to the bridge shown in this work need to be assessed and, eventually, common rehabilitation interventions are needed.

The Adige Bridge

The main lines of the Italian railway network were mostly built in the first half of the twentieth century, and the Venezia–Bologna, in the North Eastern part of Italy, at the end of the nineteenth century. The investigated structure is actually in service: it is composed by two bridges, the first in the even line (Bologna–Venezia) and the second in the odd line (Venezia–Bologna). It is important for the railway authority to assess the state of this structure, as it is the only rail line connecting these two prominent Italian cities, with freight and passenger traffic increasing year by year. A large number of such bridges are in service on the Italian railway network, so it is desirable to develop a comprehensive fatigue assessment methodology for fatigue critical details.

History, geometry and structural details

The overall bridge length is about 161 m through three spans. Simple truss spans are simply supported on the shoulders and on the central piles in the river bed. The historical bridge was built in 1866, and after 40

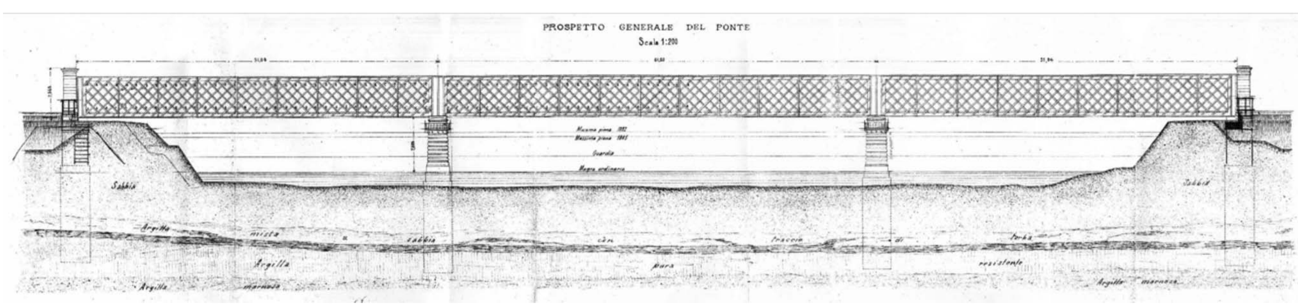

Figure 1. Adige Bridge, original configuration with single track (1866).

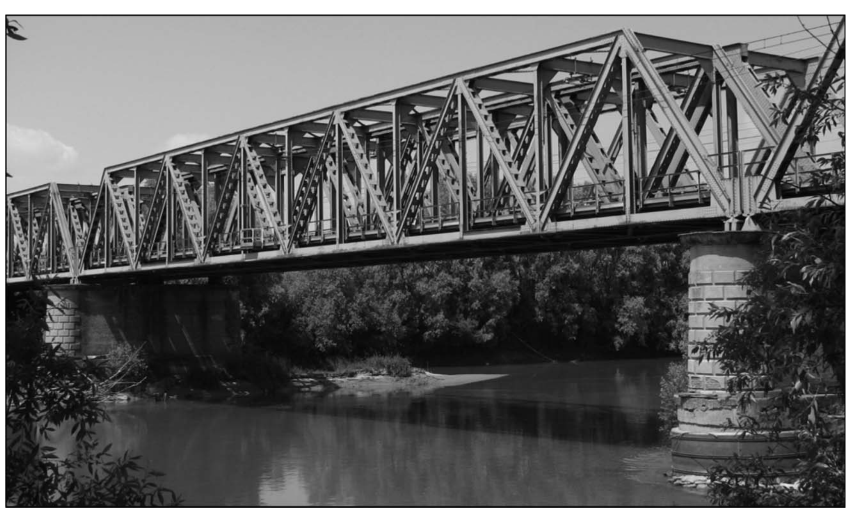

Figure 2. Adige Bridge, lateral view of the actual configuration.

years a second parallel track was completed: these bridges were both destroyed during World War II. The configuration of the bridge structure is presented in Figure 1: the even track was built in 1946 and the other one in 1949. The bridge studied is the oldest in service (from 1946).

The actual configuration is presented in Figures 2 and 3. The superstructure consists of riveted built up truss members. The bridge consists of a double three span (50.16 m–60.648 m–50.160 m $\approx$ 161 m) two-way truss girder, 5.06 m wide (from the centre of mass of the lower chords) and 7.2 m high (from lower chord to the upper one). The bridge consists of two longitudinal truss girders with transverse frame at the deck. The longitudinal truss beam is made up of 32 different cross-sections having slightly variable geometric dimensions. Lower and upper chords are composed by U-shaped sections. The deck is realised

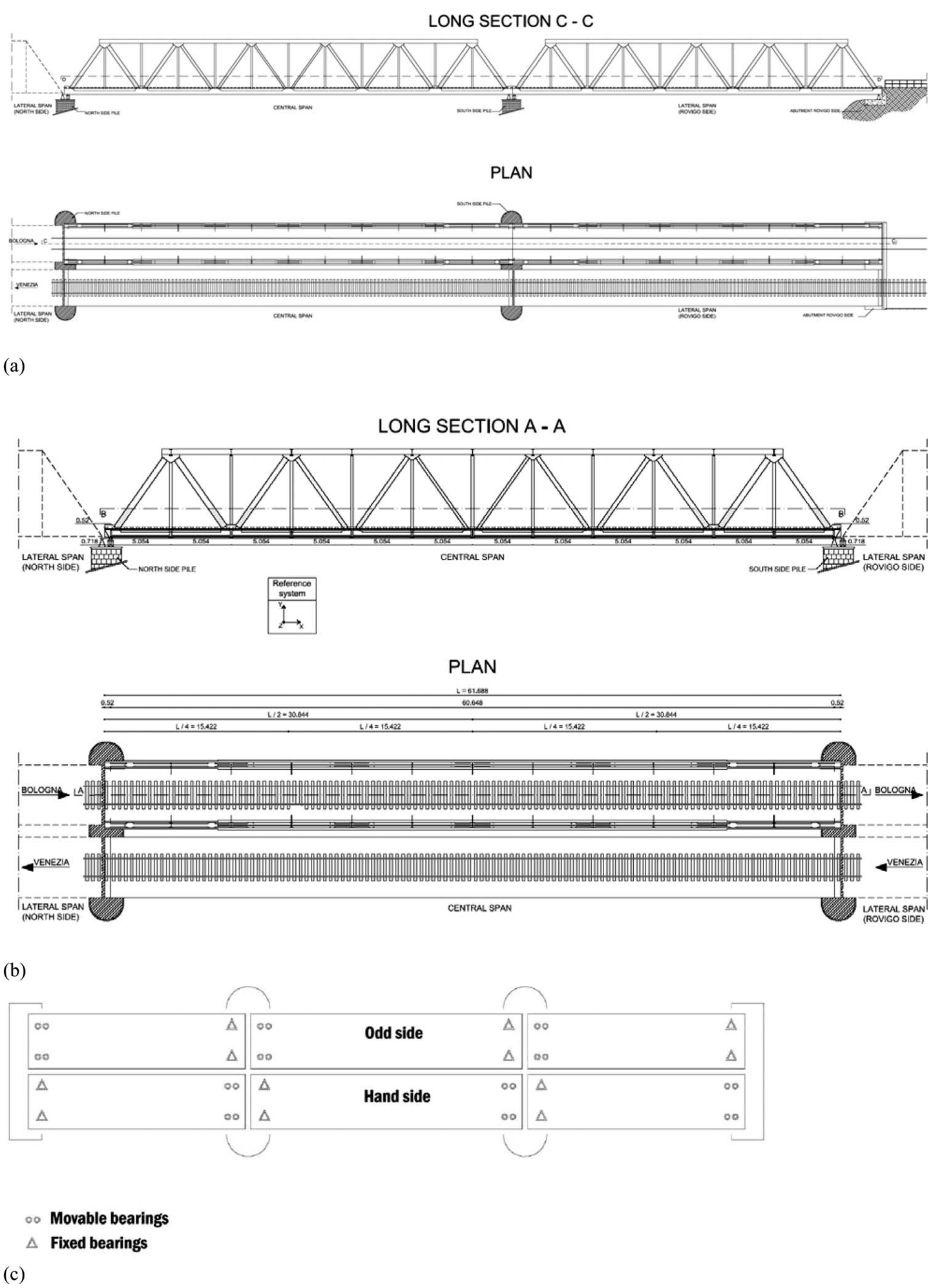

Figure 3. Adige Bridge, plan and long section of the central and south span (a); long section and plan of the central span (b); boundary conditions (c). Actual configuration.

with longitudinal stringers, and transverse floor beams. The floor beams have a fixed distance of 5.054 m, while the stringers are at 1.520 m one to the other. Stringers in their upper flange are provided for all their longitudinal length with an iron plate which is the support for the transverse ties carrying the standard gauge of 1.435 m: ties (2300 × 160 × 220 mm) are spaced at 500 mm along the bridge. Wooden transverse beams have been totally replaced along the national railway lines with concrete ones, except in metal bridges in which they stay, to avoid induced failure or cracking related to vibrations. All structural elements are built-up members, realised with plates and L-profiles with variable width from 12 to 20 mm, connected by hot riveting. Also connection joints are made of gusset riveted plates. Relevant structural details are presented in Figures 4, 5, 6. The sub-structures of every bridge are made with a 3D truss girder for torsional stiffness. Rails are of the 60UNI type. Boundary conditions are reported in Figure 3: double fixed and movable bearings stand alternately on each side span as shown; supports are presented in Figure 7.

Structure degradation

The existing structure is not severely damaged by corrosion. Maintenance works have been extended to the whole bridge, but have not been systematic. Some lower deck members are difficult to reach and consequently have not been recently re-painted: these appear black and are still in their original condition of 1946.

The design of the bridge pays attention to durability issues except for some details; for example, the U-shaped section of the lower chords and the related joints are open upwards, hence they collect large quantities of water with inadequate draining

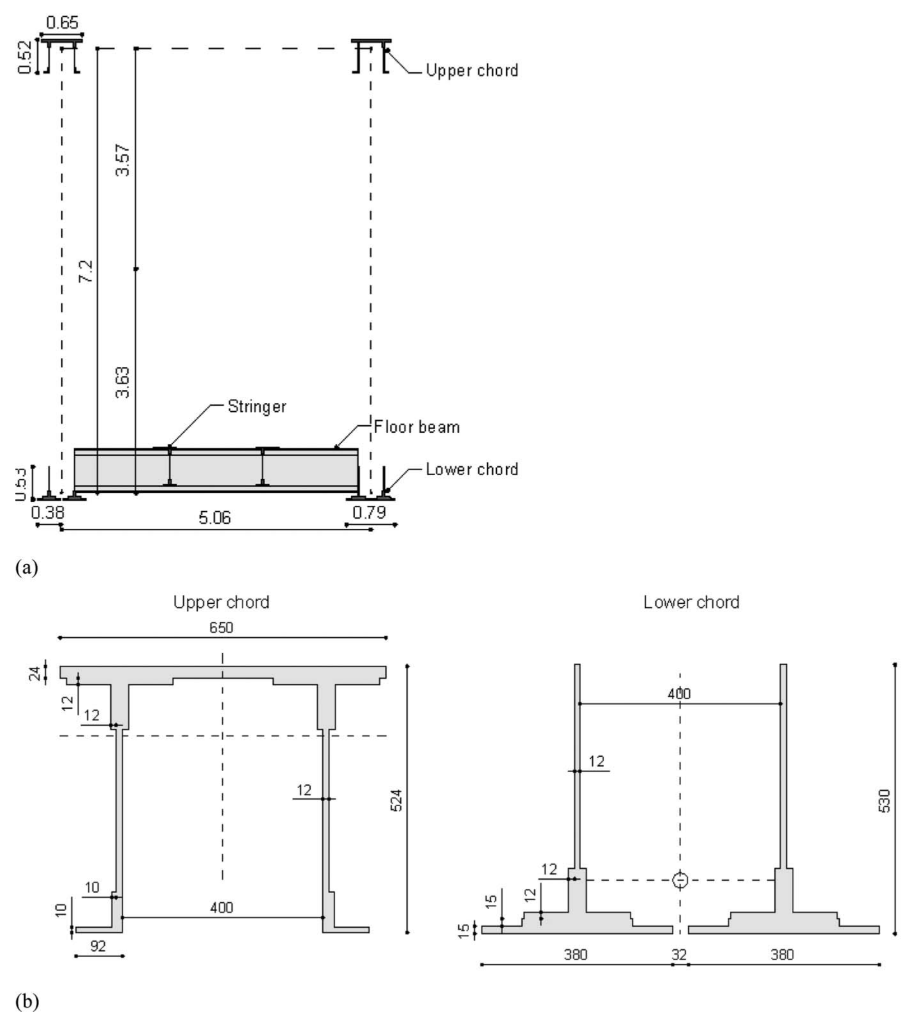

Figure 4. Adige Bridge, as built details of the reticular frame (a) and of the typical upper and lower chord (b).

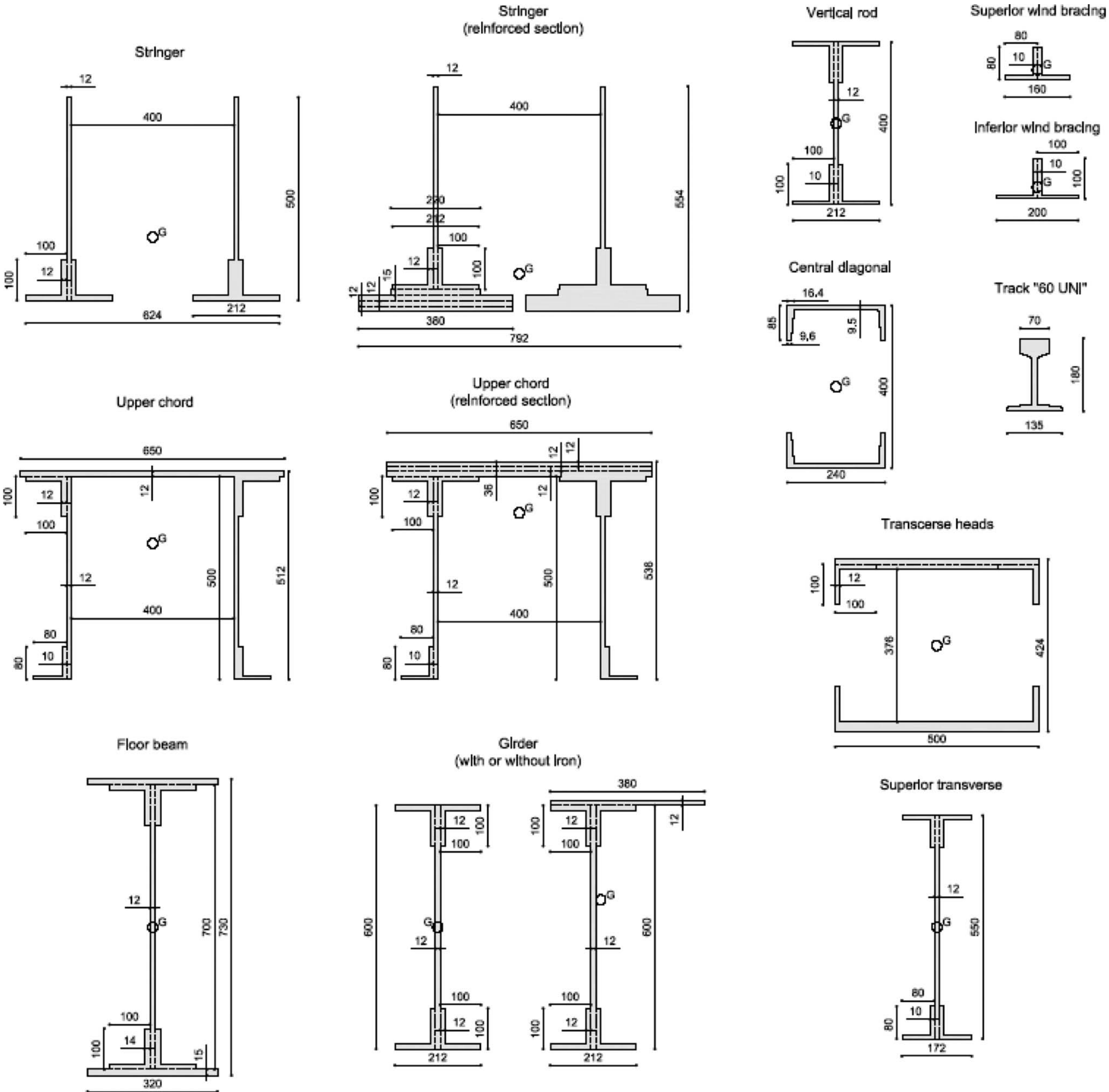

Figure 5. Adige Bridge, as built topics details.

systems. The consequence is that most of the U-shaped lower chords are affected by corrosion: the lower plate is completely corroded in some parts.

Moreover the lack of recent correct maintenance allowed corrosion to penetrate deeply inside some riveted connections leading, in some cases, to the corrosion of the rivet itself; also some loose rivets have been discovered during detailed inspections. The average depth of corrosion is approximately 1–2 mm. Corrosion is also particularly severe in the plates of the bracing system below the bridge deck and in the joints connecting the bracing system to the lower chord. Along with the detailed survey of bridge, samples were taken for laboratory testing.

Structural analysis

Material properties knowledge of existing metal bridges is essential for the assessment of the remaining lifetime (ORE 1986, Liechti *et al.* 1997, ICOM 2001, ECCS 2005, Sustainable Bridges 2007, Kühn *et al.* 2008), and this is not normally achieved in national

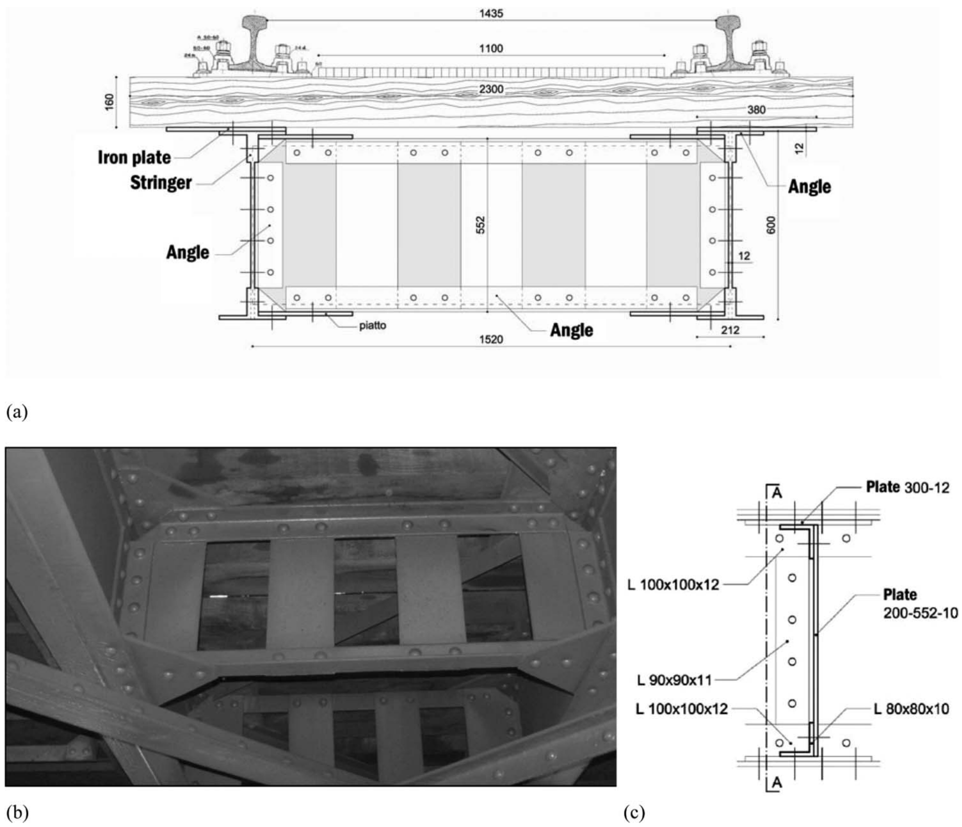

Figure 6. Adige Bridge, railway track section A-A (a), down-side view (b) and transversal section (c).

standards. Moreover, for old metal bridges built between 1870 and 1940, the material parameters are commonly not available. Sustainable Bridges (2007) suggests that the highest attention to the mechanical properties must be paid when dealing with wrought (puddle) iron and old steels, because due to the production process these irons have a large amount of slag and inclusions, plus a great anisotropy. At the same time, old steel does not usually fulfil the precise requirements of normalised materials according to EN 10025 (2004) and can have a wide variety of chemical and mechanical characteristics according to the place and the period in which they have been erected.

Material characterisation tests

Mechanical tensile tests have been developed at the Materials Testing Laboratory, Department of Structural and Transportation Engineering, University of Padova, and provided average values of 345 MPa yield strength, 442 MPa tensile strength, 31% maximum elongation and an elastic modulus of 221 GPa. Toughness from standard Charpy tests yielded 29.5 J at 20°C. According to modern standards (EN 10025, 2004), the basic material can be compared with a S275 steel (EN 1993-1-1, 2005) and the material mechanical behaviour is quite in line with steel material available at the building time. Because of the fundamental role of the rivets in the assessment procedure, as described in detail in Pipinato *et al*. (2009, 2010), some of them have been tested with the Vickers test: results highlight values very close to a Fe44B material, having an ultimate strength f_u between 430 and 530 N/mm^2 and mandatory still today in Italy for riveting (Instruction 44/T, 2000).

Structural model

The bridge structure was modelled using the FEM software Straus 7 (G + D Computing 2005), using beam and plate elements. Overall, the entire bridge model consists of about 3000 beam and 200 plate elements. A Young's modulus of 221GPa (kN/mm^2), Poisson's ratio of 0.3 and a material density value of 7800 kg/m^3 were used for the analyses. All beam members sections were modelled as the as-built structure. The FEM model of the central span has been performed as could be observed in Figure 8. One alternative model has been realised to take into account material degradation (e.g. reducing transversal section). The FEM model has been calibrated by in situ measurement.

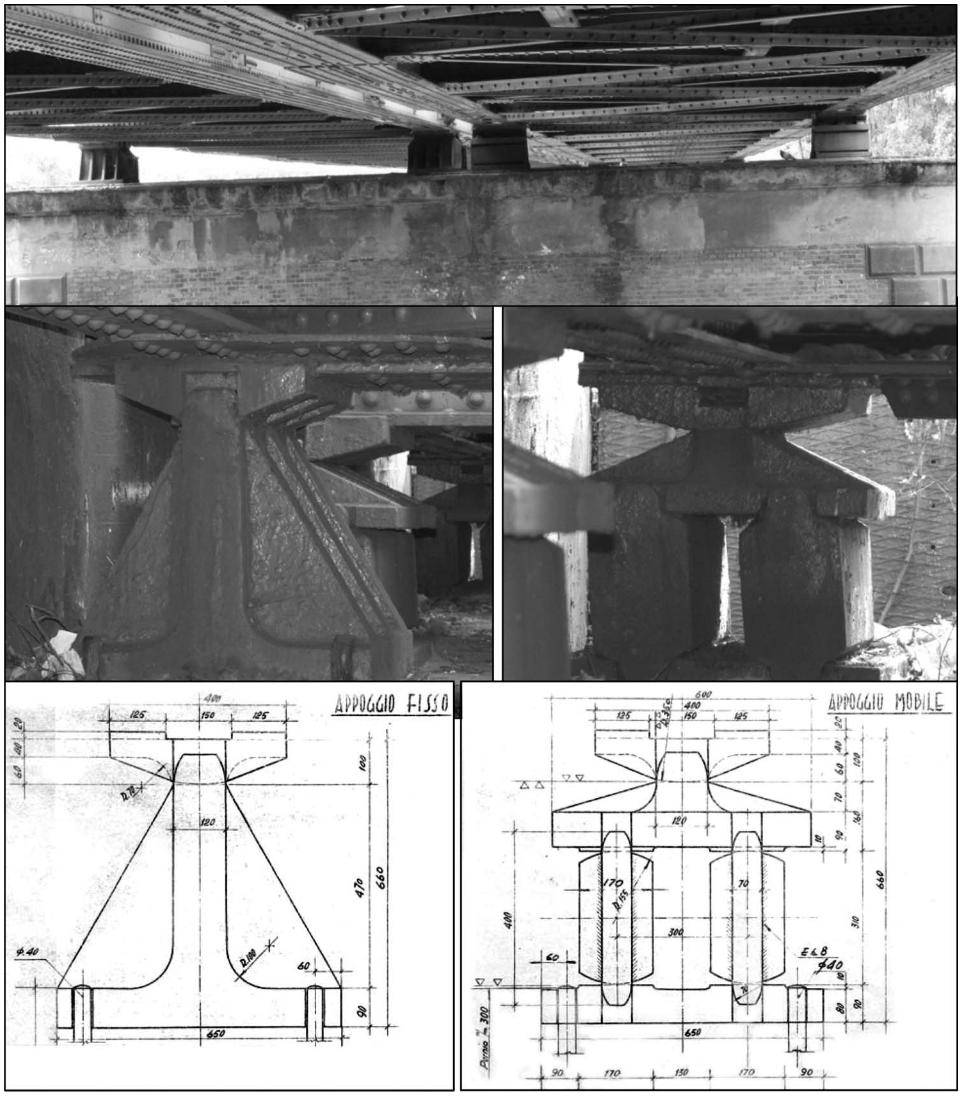

Figure 7. Adige Bridge, support details.

As the aim of the bridge assessment is referred to in the service conditions, all the models have been assumed elastic. Beam elements have been used, except for gusset and joint plates in which plates have been adopted: all member section geometry have been shaped according to the as-built dimensions.

The structural analysis has highlighted peak stresses in the stringers, hanger plates and floor beams (see Table 1). The corroded 3D frame girder model took into account a cross section reduction of 1mm extended to the whole lower chords according to the similar case study described in (Brencich and Gambarotta 2009). Results have highlighted that the structural strength is not substantially affected by this grade of corrosion. Peak values of stresses are related to the train type n. 8 freight train D4 (Instruction 44/F 1992) described in Table 2, passing in the middle of the central span. Dynamic amplification has been taken into account by using the following formula (Instruction 44/F 1992):

$$\emptyset = 1 + \varphi' + 1.5\varphi'' \tag{1}$$

where

$$\varphi' = \frac{\mathrm{k}}{(1 - \mathrm{k} + \mathrm{k}^4)}$$

$$\mathrm{k} = \frac{\mathrm{v}}{(2\mathrm{L}\mathrm{n}_0)}$$

$$\eta_0 = \left(\frac{438,8}{\mathrm{L}}\right)^{(1/1,33677)}$$

for bridges along main lines,

$\varphi'' = 0.01\alpha\{56\ \mathrm{e}^{-(\mathrm{L}/10)2} + 50\,[n_0\,\mathrm{L}/80]^{-1}\}\ \mathrm{e}^{-(\mathrm{L}/20)2}$

v = train speed

$\alpha = 1$ for $\mathrm{v} > 22$ m/s

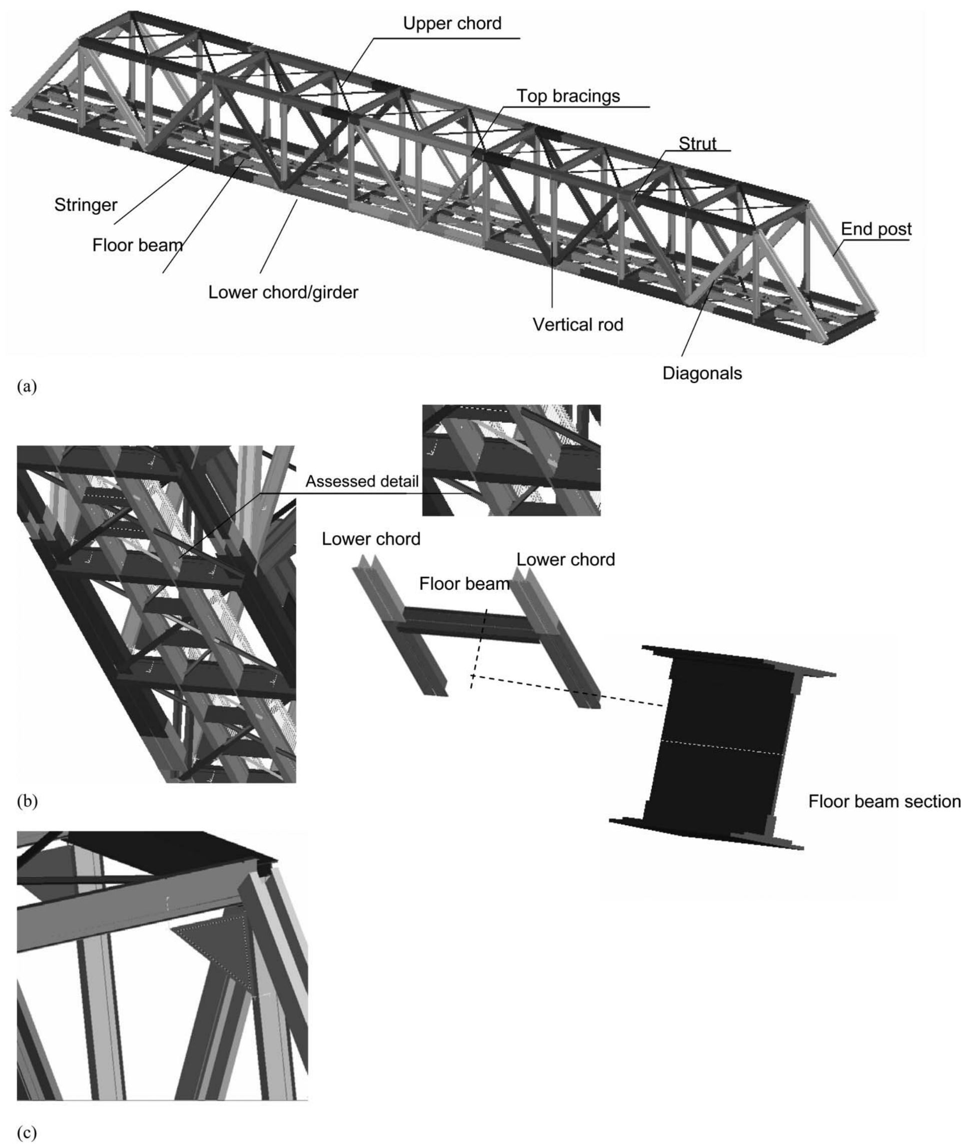

Figure 8. Adige Bridge, FEM 'as built' model of the central span (a); detail of the floor beams and of the detail assessed (see Figure 9) (b); and of the connection between plate elements and beam elements, which are connected with rigid link (c).

Table 1. Extreme values of the stresses in the structural members from the different models (Figure 8). Results refer to train type 8 (see Table 2).

Structural model	Stresses [MPa]					
	Stringer		Hanger plate		Floor beam	
	Max	Min	Max	Min	Max	Min
3D girder as built	102	11	140	14	99	12
3D girder corroded	+1%	+1.2%	+1.3%	+1.1%	+1.2%	+1.3%

where v is the maximum train speed,

$$L = \text{span [m]}$$

Fatigue assessment and remaining life analysis

For the period starting from construction to 1990, a historical load model was used. For the actual load traffic Instruction 44/F (1992) was adopted: this code provides for each train type the number of train/day, the corresponding total number of axle/day, the locomotive and carriages denomination, their t/axle load, the corresponding number of axles and finally the quantity of every carriage (Table 2). The historical load spectrum is represented by 13.2 t/m locomotives and 8 t/m carriages from 1945 till 1990. All trains consist of an engine car placed in front followed by a series of wagons. Fatigue assessment has been performed for the oldest bridge central span applying the load steps statically, at Service Limit Fatigue State

Table 2. Instruction 44f (1992): daily traffic spectrum and train loads for fatigue verification.

Train type	Name	Train/day	Axle/day	Locomotive (L) and carriages (Ci)	t/axle	Wagon number	Wagon type	Axle spacing [m]
1	IC	20	960	L	20, 25	1		2.6–6.4–2.6
	Intercity			C1	15	5		2.56–16.44–2.56
				C2	12, 75	6		2.56–16.44–2.56
2	EC	10	340	L	20	1		2.85–2.35–2.85–2.35–2.85
	Eurocity			C1	14, 25	2		2.56–16.44–2.56
				C2	12	5		2.56–16.44–2.56
3	EXPR	15	990	L	20	1		2.85–2.35–2.85–2.35–2.85
	Express			C1	14, 25	10		2.56–16.44–2.56
				C2	12	5		2.56–16.44–2.56
4	DIR	30	1380	L	18,6	1		2.85–2.35–2.85–2.35–2.85
	Direct			C1	10, 675	10		2.4–16.6–2.4
5	ETR	10	480	L	20	2		3–9–3
	Eurostar			C1	11, 6	10		3–17.3–3
6	TEC	15	990	L	18, 7	1		2.85–2.35–2.85–2.35–2.85
	Container freight			C1	20	15		1.8–12.8–1.8
7	Merci acciaio	10	720	L	18, 7	2		2.85–2.35–2.85–2.35–2.85
	Steel freight			C1	20	15		1.8–13.06–1.8
8	Treno merci tipo D4	5	380	L	20	2		2.85–2.35–2.85–2.35–2.85
	D4 freight			C1	22,5	16		1.8–4.65–1.8
9	Treno merci misto	5	270	L	18, 7	1		2.85–2.35–2.85–2.35–2.85
	Mixed freight			C1	16	24		9

(SLFS) basing on Instruction 44/F (1992). Dead loads of the bridge and superimposed dead load (sleepers and rails) are computed by the FEM program. All standard trains provided by Instruction 44F (1992) have been analysed in several positions in order to find the critical details.

The damage caused by the passage of a single train was calculated according to Instruction 44/F (1992) first by using the rain-flow counting method to convert the irregular stress history into stress range blocks, and then by applying the Palmgren (1924) and Miner (1945) rule.

The cumulative damage approach implies the use of the formula:

$$D_{d,EC_a} = \sum_{i}^{n} \frac{n_{E_i}}{N_{R_i}} \leq 1.0, \tag{2}$$

where n_{E_i} is the number of cycles associated with the stress range $y_{F_f}\Delta\sigma_i$ for band i in the factored spectrum [MPa]; $N_{R_i}$is the endurance (in cycles) obtained from the factored $\frac{\Delta\sigma_C}{y_{M_f}}$ vs. N_R curve for a stress range of $y_{F_f}\Delta\sigma_i$ [MPa], $\Delta\sigma_C$ – reference value of the fatigue strength at $N_C = 2$ mil cycles [MPa]; y_{M_f}– partial factor for fatigue strength $\Delta\sigma_C$.

The constant amplitude damage was calculated by adopting the two-slope S–N curve of Instruction 44/F (1992) shown in Figure 10. The investigation was carried out for the critical riveted details reported in the following.

Fatigue remaining life assessment

According to the structural analysis and in agreement with recent studies (Akesson and Edlund 1996, Al-Emrani 2000, 2005) critical details of the structure resulted to be:

- the joint connection between vertical floor-beam tension hangers and horizontal top compression bars;
- the stringer-to-floor beam connections.

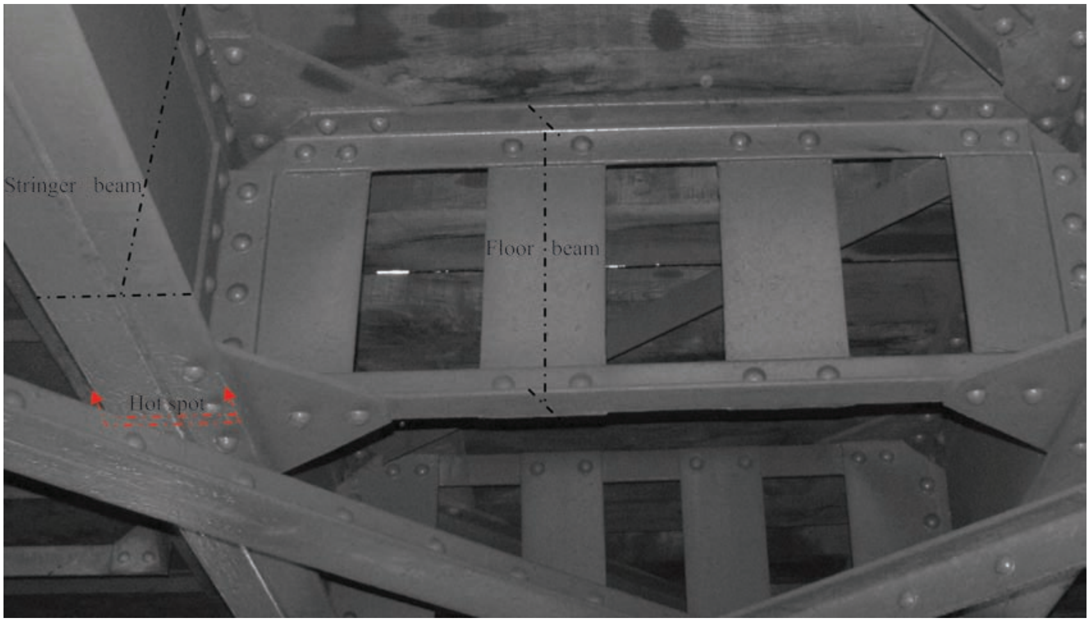

Figure 9. Adige Bridge detail of the flange assessed.

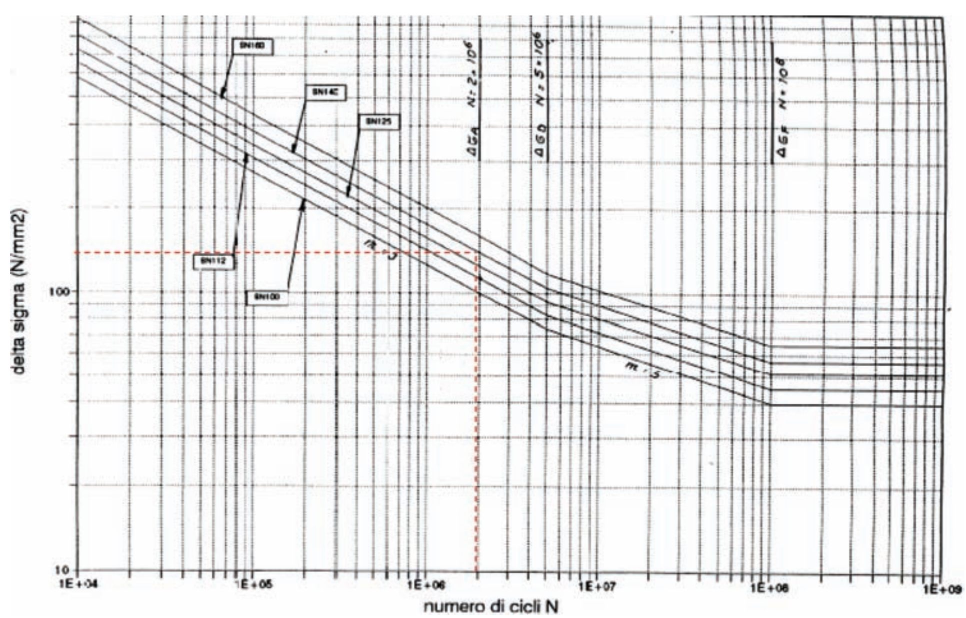

Figure 10. SN curve for structural details subjected to compression or tensile stress according to Instruction 44/F(1992).

Assuming the dead load as a reference state, the effect of load cycles is represented by the stress ranges $\Delta\sigma$ induced by trains calculated at the serviceability limit state – frequent loads (EN 1993-1-9 2005).

The following discussion will be focused only on the fatigue assessment of the stringer-to-floor beam connection, in detail at the lower flange of the floor beam where the stringer is jointed to the floor beam (Figures 8 and 9), being at the same time the critical detail with the unfavourable combination of stress value and suffering the highest number of cycles for the train spectrum considered. Secondary stress fluctuations are far below the limit for cumulative damage.

Assessment results are reported in Table 3: this provides for each period (1946–1990, 1990–2008 and future) and each train type, the equivalent $\Delta\sigma_e$ of the assessed critical detail, the number of cycles n_i performed for each year, the SN curve slope m adopted in calculations and the cumulated damage. According to these results, a precise estimation of the

Table 3. Remaining fatigue life prediction.

Period 1946–1990	Type of train	$\Delta\sigma_e$ [MPa]	ni	m	ni/Ni
	Historical train	59.84	1,606,000	5	0.306
				SUBTOTAL	*0.306*
Period 1990–2008	Train 1	90.67	131,400	5	0.090
	Train 2	90.67	65,700	5	0.044
	Train 3	90.67	98,550	5	0.065
	Train 4	84.32	197,100	5	0.105
	Train 5	90.67	65,700	5	0.044
	Train 6	84.77	98,550	5	0.053
	Train 7	84.77	65,700	5	0.036
	Train 8	91.80	32,850	5	0.022
	Train 9	84.77	32,850	5	0.018
				SUBTOTAL	*0.477*
Future 10 years (no increase of traffic)	Train 1	90.67	73,000	5	0.050
	Train 2	90.67	36,500	5	0.024
	Train 3	90.67	54,750	5	0.036
	Train 4	84.32	109,500	5	0.058
	Train 5	90.67	36,500	5	0.024
	Train 6	84.77	54,750	5	0.030
	Train 7	84.77	36,500	5	0.020
	Train 8	91.80	18,250	5	0.012
	Train 9	84.77	18,250	5	0.010
				SUBTOTAL	*0.265*
Future 9 years (+5% increase of traffic)	Train 1	90.67	68,985	5	0.047
	Train 2	90.67	34,493	5	0.023
	Train 3	90.67	51,739	5	0.034
	Train 4	84.32	103,478	5	0.055
	Train 5	90.67	34,493	5	0.023
	Train 6	84.77	51,739	5	0.028
	Train 7	84.77	34,493	5	0.019
	Train 8	91.80	17,246	5	0.011
	Train 9	84.77	17,246	5	0.009
				SUBTOTAL	*0.250*
Future 8 years (+10% increase of traffic)	Train 1	90.67	64,240	5	0.044
	Train 2	90.67	32,120	5	0.021
	Train 3	90.67	48,180	5	0.032
	Train 4	84.32	96,360	5	0.051
	Train 5	90.67	32,120	5	0.021
	Train 6	84.77	48,180	5	0.026
	Train 7	84.77	32,120	5	0.017
	Train 8	91.80	16,060	5	0.011
	Train 9	84.77	16,060	5	0.009
				SUBTOTAL	*0.233*
Future 7 years (+15% increase of traffic)	Train 1	90.67	58,765	5	0.040
	Train 2	90.67	29,383	5	0.019
	Train 3	90.67	44,074	5	0.029
	Train 4	84.32	88,148	5	0.047
	Train 5	90.67	29,383	5	0.019
	Train 6	84.77	44,074	5	0.024
	Train 7	84.77	29,383	5	0.016
	Train 8	91.80	14,691	5	0.010
	Train 9	84.77	14,691	5	0.008
				SUBTOTAL	*0.213*

remaining life of the bridge, focusing on four trends of the type of future traffic demands, with an increment of the 5%, 10% and 15% for each train type is given; no increase in loads has been taken into account. The fatigue category of the critical detail investigated is C = 140, according to the Instruction 44/F (1992). Fatigue category C = 140 deals with not-welded riveted gas cut detail (Instruction 44/F, 1992). The fatigue curve adopted in this study is the SN-curve compulsorily used in Italy and given in the code Instruction 44/F (1992). Some comparisons among different SN curves for riveted structures have been included in recently published works by the authors (Pipinato *et al.* 2009, 2010) to compare the Italian curve with the curves given by some International codes. Probably fatigue strength of 80 MPa at 2 million

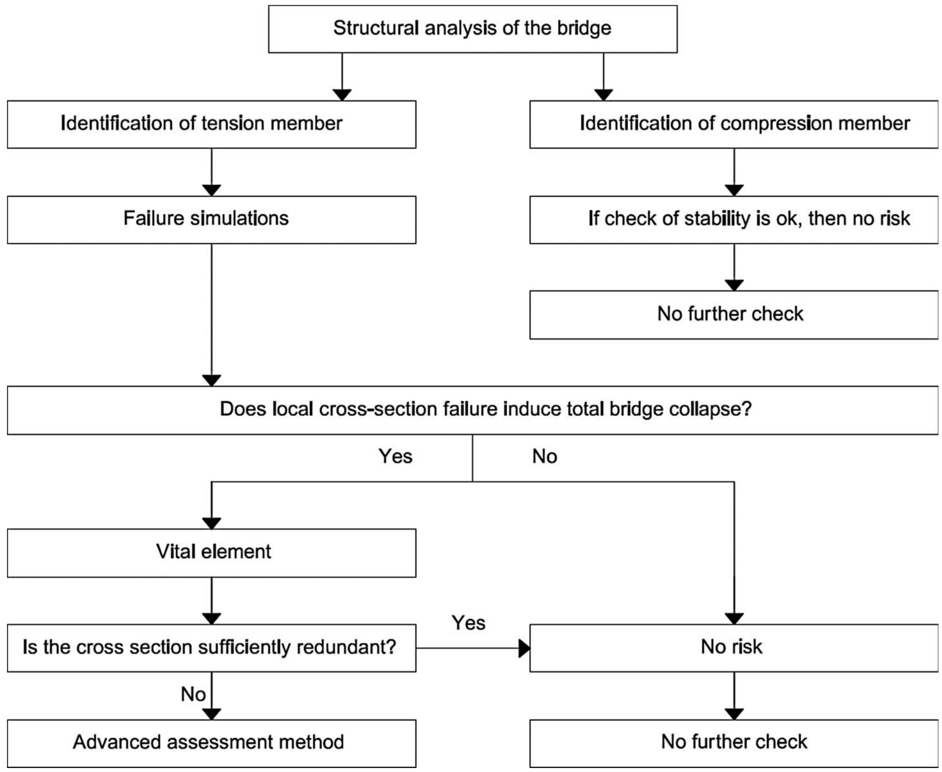

Figure 11. Procedure for identification of vital elements.

cycles could be more realistic but the Italian code considers 140MPa for this case and the paper aims to show the procedure to assess a bridge of the Italian Railway Authority.

The estimated residual life of the bridge could be inferred from Table 3 according to the Palmgren (1924) and Miner (1945) rule, and is herein summed up by highlighting the remaining life:

- 10 years with the same actual traffic.
- 9 years with an increment of the 5% of traffic from 2009.
- 8 years with an increment of the 10% of traffic from 2009.
- 7 years with an increment of the 15% of traffic from 2009.

Since the remaining fatigue life is no much longer, the main issue pertaining to ensure the service of the bridge is a complete maintenance and retrofitting program aiming at stopping the effects of corrosion and possible fatigue cracks initiation or propagation on the structure.

Retrofitting procedure: intervention criteria

Retrofitting procedures are needed to strengthen bridge structural and non-structural members: in the following, a general damage assessing procedure is presented. The most extended phenomenon generally affecting these type of old metal railway bridges is corrosion, that might have occurred on the bridge due to harsh environmental conditions and the loss of design accuracy to avoid water stagnation.

Being corrosion the most common damage issue affecting these structural details, it is relevant to mention the most common details affected by corrosion, such as:

- lacing bars;
- pinned/riveted connections, where small relative movements might trap moisture;
- bottom flanges of members due to debris build-up;
- top flange of floor or girder beams due to deck leakage.

Fatigue capacity is related to corrosion damage and if the corrosion loss was less than 50% of the initial resisting area, notch effects rather than section loss governed fatigue capacity (Thiel *et al.* 2001). Members to be investigated has to be chosen as shown in Figure 11 according to ESDEP (2007) stepwise procedure. In the following, solutions against corrosion are given:

- *Do nothing solution:* If section loss is under the 15%, only NDT (non-destructive tests) are necessary in order to investigate the ultimate strength of the structural element; a simplified FEM model could be performed to check the corroded structure in the critical details. If corrosion is diffused to the main structures of the bridge, even if section loss is under the 15%, a prevention of corrosion through painting works is needed: in this case, the precise intervention decision could be quantitatively assessed considering chloride contamination according to Chong (2004).
- *Repair member solution:* If section loss is under the 40% of the initial resisting area, hence the corroded member is moderately deteriorated, and a repair of the member might be warranted. Rehabilitation technique is strictly related to the type of member: for example, a corroded tension member could be strengthened by adding new steel plates, a compression member can be reinforced using cover-plates, or by post-tensioning. As a reference, some cover-plated and bolted repairs for members of the Eads St. Louis bridge (Luis Silano 1992) whose floor system had been badly deteriorated by corrosion, are reported in Figure 12: the weakened area is reinforced by cover-plating and bolting, or only by bolting in a rivet failure case.
- *Replace member:* If section loss is over the 40%, the corroded member is severely deteriorated. Removing and replacing a member include the following stages:
  - (i) support structural system before removing member;
  - (ii) remove damaged member;
  - (iii) add replacement member;
  - (iv) remove supporting system.

  In Figure 13 could be found the outlines of the replacement of a diagonal tension member of a truss bridge similar to the one investigated according to Luis Silano (1992).
- *Replace the entire structure:* This choice has to be adopted in situations in which member replacing or repairing is not affordable for time, traffic delay or cost issues.

A detailed procedure on retrofit or structure replacement can be found in Kühn *et al.* (2008), while a global cost analysis should be performed according to BRIME (2001) to afford the more suitable intervention.

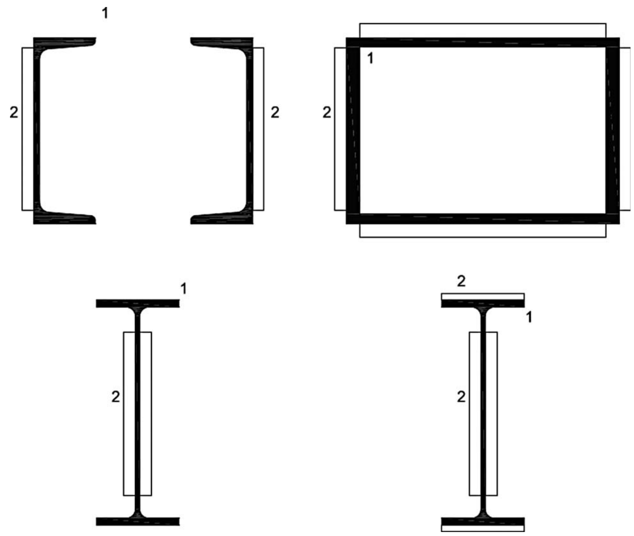

Figure 12. Bolted repairs for corroded members: new member (1), existing eyebar (2).

Retrofitting procedure: strengthening of structural members

Historical bridges are often under-strength or have to be retrofitted to carry new loading provided by standard innovation or to improve its fatigue resistance. These bridges were often built before the development of national standards, or designed using past loading models. If the entire structure is inadequate, its replacement should be adopted. On the other hand, if one or a few members are under-strength, then rehabilitation might address only those members. Herein structural member strengthening techniques are presented as possible guidelines for common design situations, in particular for floor beams, girders and stringers, tension member, compression member, pinned connections and riveted connections.

- *Cover plating:* Cover plates are either bolted or welded to the existing member to increase the rigidity of the member, thereby decreasing the stresses present in the member. Again, welding of old metals should be adopted with caution. Welding might cause delamination of wrought iron or fatigue cracking at the ends of cover plates.
- *Post-tensioning:* This technique relies on a supplementary element, e.g. to apply a negative moment, thereby reducing the flexural stresses in the member. Post-tensioning bars or pre-stressing tendons can be often used to apply equivalent external forces to the system. These systems increase the allowable service loads in the member.

- *King post:* The principles of post-tensioning is utilised, but applied with a different geometry. King posts form a triangular shape stemming from a bracket located at midspan of the beam separating the tendon from the flexural member. The primary benefit of the king post is the small axial force in the tendon, relative to the high negative moment applied to the beam. An example of king-post is reported in Figure 14 (Luis Silano 1992).
- *Composite action:* It provides strengthening by adopting a composite deck solution, in which shear is transferred between concrete slab and steel beams.
- *Additional members:* The new members may redistribute the forces to other bridge elements. The addition of new members is most commonly undertaken during a deck replacement. See, for example, Figure 15 according to Klaiber (1987).
- *FRP strengthening:* A recent technique for strengthening steel structures consist of the application of externally bonded fibre reinforced polymer (FRP) sheets, mainly used to increase tensile and/or flexural capacity (Figure 16) of the structural element (Pellegrino *et al.* 2009). FRP materials have a high strength to weight ratio, do not give rise to problems due to corrosion and are extremely manageable. Some examples of guidelines for the design and construction of externally bonded FRP systems for strengthening existing metal structures are the ICE design and practice guide (Moy 2001), CIRIA design guide (Cadei *et al.* 2004), US Design Guide (Schnerch *et al.* 2007) and CNR-DT 202/2005 document (Italian Research Council 2005). The benefits of composite strengthening have been applied, for example, in a steel bridge on the

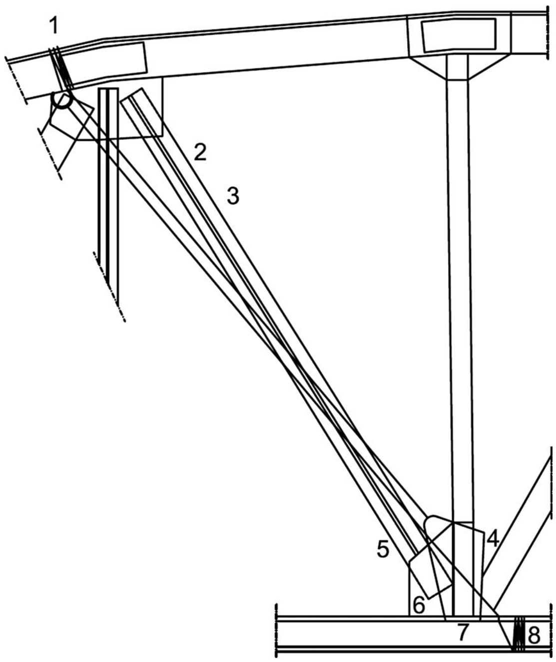

Figure 13. Replacement of diagonal tension member: (1) cable with minimum four wraps, (2) upper diagonal to be replaced, (3) one loop cable and one turnbuckle, (4) blocking for cable to clear flange of floor-beam, (5) cable with minimum four wraps, (6) burn holes in lateral plate for cable, (7) half round wood block or similar steel specimens, (8) cable with minimum four wraps.

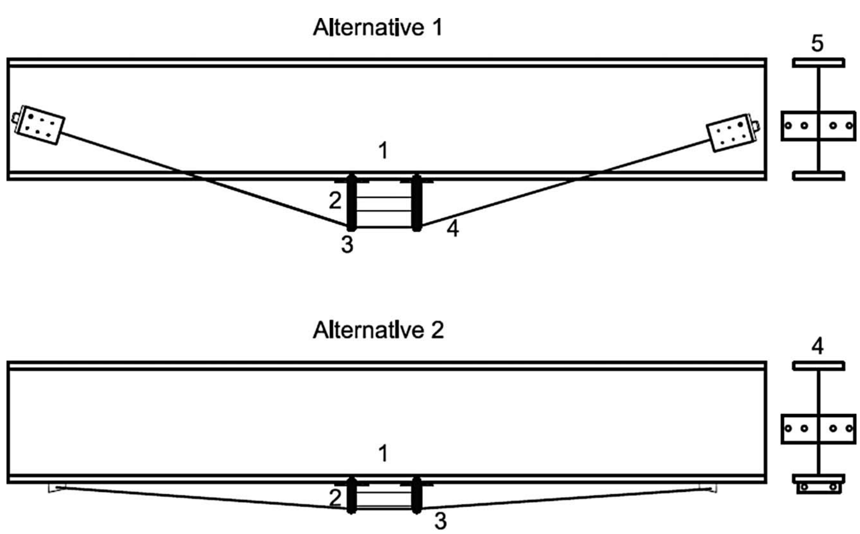

Figure 14. King post strengthened beam technique intervention, two alternative interventions. Alternative 1: existing beam (1), king post support (2), rounded edges on the bottom of the webs to prevent corner bearings (3), post tensioning truss rods (4), current section (5). Alternative 2: existing beam (1), king post support (2), post tensioning truss rods (3), current section (4).

London Underground (Moy and Bloodworth 2007). The benefits of strengthening large cast-iron struts with carbon FRP composites in the London Underground are illustrated in Moy and Lillistone (2006). A state-of-the-art review on FRP strengthened steel structures was recently developed by Zhao and Zheng (2007).
- For truss railway bridges, the critical structural members to be strengthened could be:
- *The main compressive members:* These are generally repaired through cover plating as reported in Figure 17 according to Bondi (1985).
- *The riveted connections:* They are often susceptible to debris and corrosion due to the built up nature of the elements; furthermore it has to be considered that during the original design operation, fatigue may not have been considered, whereas a connection repair should address fatigue and designed accordingly (Brühwiler *et al.* 1990). High strength bolts should be used to replace rivets; this intervention introduces a higher clamping force imposed by the new bolts acting as an increase in shear capacity and fatigue life. If corrosion or cracking propagate on the main parts of the member investigated, member replacing should be the most suitable intervention.

Retrofitting proposals for the Adige Bridge

As a result of the aforementioned analysis, due to the high degree of redundancy discovered both in the structure as a whole and in the joints, the Adige Bridge is not in as dramatic a condition as it would first appear. No welding works are scheduled. Localised severe corrosion, and extended superficial damage lead to the conclusion that superficial treatment and paintings on trusses would be necessary to prevent higher section loss than the actual situation. Other retrofitting works will be related to:

- Accurate repainting of internal part of lower chords.
- Replacing of the most corroded elements as hangers and bracings.
- Local repairing of floor beams throughout low-flange cover plating.
- Replacing damaged riveted connections with high strength bolts and cover plating works on connecting plates.

These works should be carefully adopted only after a deeper assessment of the bridge, and if cracks will be discovered in critical details, for example, in the flange of the net cross section, repair should be addressed by removing cracked members or adding longer cover and filler plates in the tension flange with bolts, using high strength bolts in the last connection of the upper cover plate, or adding FRP laminates.

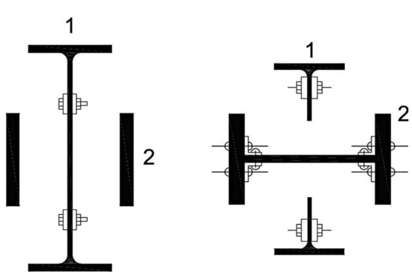

Figure 15. New member added to existing tension members: new member (1), existing member (2).

Conclusions

In the framework of the general necessity of the authorities managing railway lines to assess and, eventually, strengthen existing steel bridges, the assessment of the Adige Bridge, included in a main railway line in Italy and having a common scheme for this kind of structures, has been developed by means of a stepwise procedure consisting in:

(i) mechanical characterisation of the materials;
(ii) survey of the damaging effects;
(iii) structural analysis,
(iv) fatigue assessment together with traffic estimations,

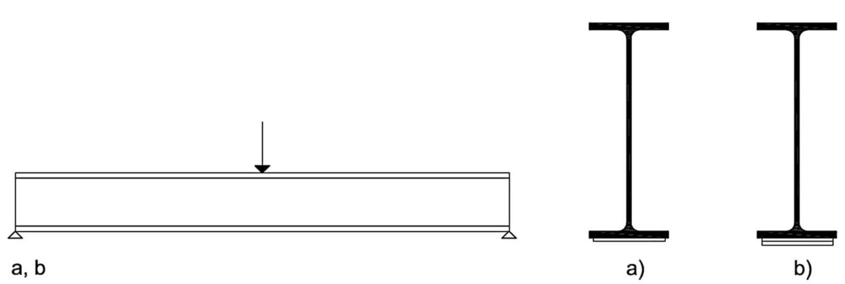

Figure 16. FRP strengthening of a steel flexural element.

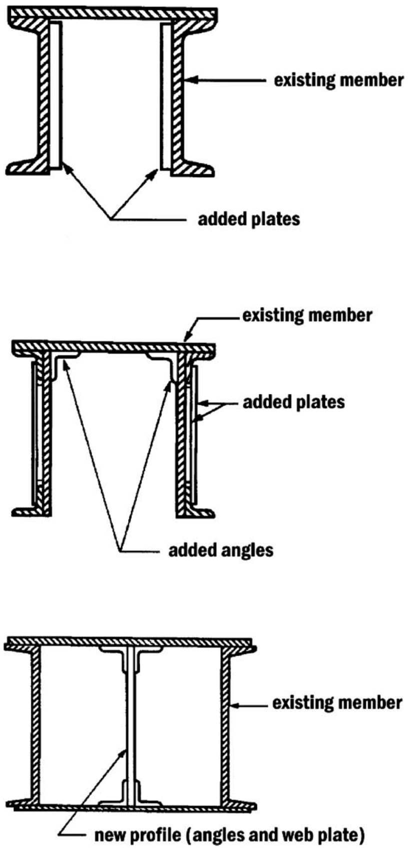

Figure 17. Cover plate options for compressive members: existing members (1) and new members (2).

(v) review of the most common intervention techniques related to historical riveted steel bridges,
(vi) proposals of the retrofitting works.

The application of such a procedure to the Adige Bridge outlined some relevant issues:

- The materials used in this quite old railway bridge have shown mechanical and chemical properties (strength, chemical composition) comparable to those provided by actual Eurocodes (an adequate material characterisation is recommended for this type of bridges, since similar materials, but with different mechanical and chemical characteristics, could be used in relation to the place and the period in which the structure was built).
- Corrosion is a local phenomenon that does not necessarily play a relevant role on the global response of the bridge if section loss is less than 15%.
- Fatigue cracks, not discovered in this first phase assessment, have to be properly taken into account in a deeper assessment phase, carefully monitoring structural details as floor hangers, stringer-to-floor beam connections, short diaphragms and riveted connections.
- Fatigue assessment concerning the remaining life could be performed using the Palmgren (1924) and Miner (1945) rule, accounting the possible traffic increase for the estimation of the residual lifetime: the result gives a reasonable and detailed estimation of safe exercise.
- Monitoring should be an option when the 'calculated' fatigue safety is insufficient.

The application of such a comprehensive procedure to the Adige Bridge, with a common scheme for railway bridges, showed that a detailed analysis and assessment can result in limiting the economical effort for retrofitting works to a small fraction of the costs needed for replacing the bridge.

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