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High-cycle fatigue behavior of riveted connections for railway metal bridges

A. Pipinato [a,*], C. Pellegrino [a], O.S. Bursi [b], C. Modena [a]

[a] Department of Structural and Transportation Engineering, University of Padua, Italy

[b] Department of Mechanical and Structural Engineering, University of Trento, Italy

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ABSTRACT

The evaluation of existing structures, in particular bridges, is becoming increasingly important. In particular for railway bridges, service loads and stress cycles accumulated under traffic loads, and the consequent ageing of existing structures, lead to the need for an assessment of their remaining fatigue life, in order to decide on retrofit or structure replacement. In this context, a 12.4 m span railway bridge near Sacile, Italy, with a common structural scheme for railway bridges, about ninety years old, was taken out of service, transported to a structural laboratory and subjected to both material characterization, monotonic and high-cycle fatigue shear tests. Materials exhibited a yield strength f_y and a tensile strength f_u of about 322 and 421 MPa, respectively, and the hot-spot critical details resulted in the riveted connections of the shear diaphragms that carried the rails. Other material properties that affected the fatigue endurance in a favorable way, and are not taken explicitly into account in structural Codes, were determined. As a result, the equating of riveted shear splices with splices jointed with non-preloaded bolts has been shown, and Eurocode 3-1-9 [EN 1993-1-9. Eurocode 3: Design of steel structures — part 1-9: Fatigue, Brussels: CEN; 2005] design rules have resulted on the safe side, even though no specific category of riveted details has been found to be available.

1. Introduction

In recent years, the increment of mobility and traffic on transport infrastructures has been leading to an increase of both load and speed on bridges. In particular in railways, bridges represent a strategic part of an ancient network and, in several cases, they have already reached their traffic capacity limit. In this context, bridge condition state assessment and consequently maintenance/replacement operations become more and more necessary. The average age of sixty percent of Italian railway steel bridges is about one hundred years as they were built between 1900 and 1920. Moreover, a study of the ASCE Committee on Fatigue and Fracture Reliability [1], pointed out that eighty to ninety percent of failures in steel structures were to be related to fatigue and fracture.

With regard to fatigue assessment of riveted historical metal bridges, many factors have been found to play an important role, as also documented by several studies (see among others: Bruhwiler et al. [2], Kulak [3], Di Battista et al. [4], Bursi et al. [5], Matar and Greiner [6], Pipinato [7], Pipinato et al. [8]). With regard to loadings, the dead load vs. live load ratio is usually between 15%–20%, thus entailing 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 assessment. Vibrations, transverse horizontal forces, internal constraints, localized and diffused defects such as corrosion damages, represent concurring causes of fatigue damage. In addition, the use of different riveting techniques either in-shop or on-site may entail different clamping force levels and variable load-carrying capacities both in members and in joints.

Bruhwiler et al. [2] developed extensive tests concerning fatigue strength of rivets under shear loading. Concerning this specific topic, this author suggested that failures occurred first due to shear stresses within rivets, and that such rivet failures could be probable because riveted connections were often proportioned according to the dimensions of the elements in the connection, rather than designed using allowable stresses.

We present in this paper a series of experiments and code comparisons on an old railway riveted metal bridge taken out of service and transported to a laboratory of the University of Padua. In the first phase, a code comparison between the shear-type literature data and fatigue design curves provided by Eurocode 3-1-9 [9] was performed. The focus was placed on shear riveted connections of short diaphragm that carried the rails. In the second phase, a material characterization of the aged constitutive materials was carried out. The factors that could exert an influence on the fatigue endurance were observed since they

* Corresponding author.
E-mail address: alessio.pipinato@unipd.it (A. Pipinato).

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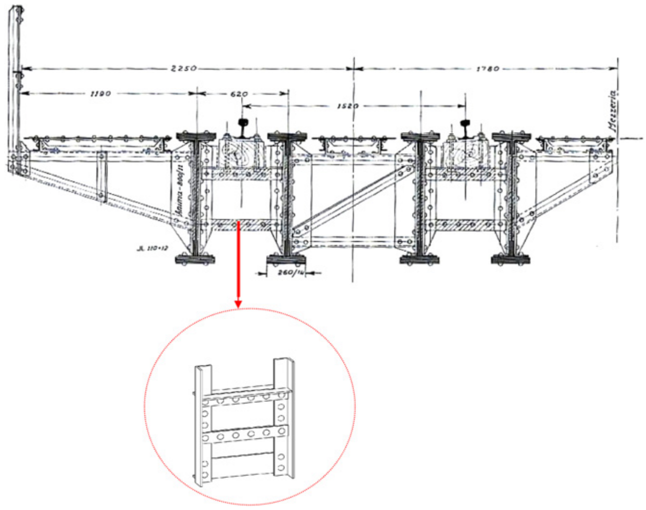

Fig. 1. Short diaphragm connections and its larger view.

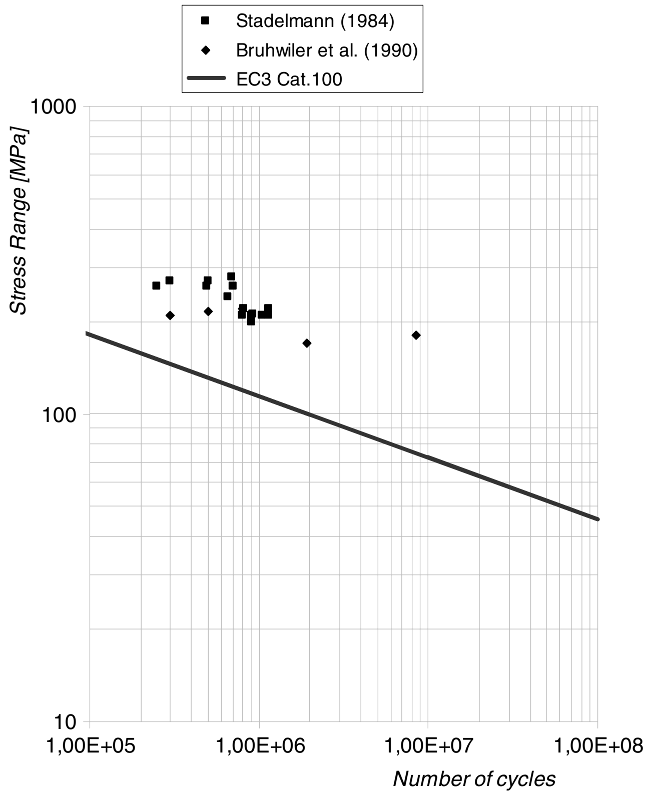

Fig. 2. Test data and fatigue design curves, Shear Category 100 (Eurocode 3-1-9, 2005).

are not explicitly taken into account in codes. In the third and last phase, four high-cycle shear fatigue tests on short diaphragm shear riveted connections were presented and compared with Category 100 of Eurocode 3-1-9 [9].

The literature studies concerning the topic of riveted railway bridges, generally approach the matter by:

- using fracture mechanics methods in order to predict crack sizes, in view of estimating information about the remaining fatigue life (Kühn et al. [10]);
- adopting Finite Element (FE) approaches to investigate in depth the stress distributions in complex connections and rivets (Al-Emrani M. and Kliger [11]);
- exploiting FE analyses in order to perform 3D studies on statically indeterminate bridges (Brencich and Gambarotta [12]).

Because, materials, details, joints and structures are generally complex, all the aforementioned model-based studies need experimental validations.

Conversely, the objective of this study was different: to investigate the classification method based on S–N curves, in order to assess the shear Category of EN 1993-1-9 [9] for riveted shear splices. Because the examined bridge was characterized by a simple structure, i.e. a statically determinate bridge, and the shear category is based on nominal stresses, it was rather easy to evaluate the nominal shear stress on riveted connections of the short shear diaphragms that carried the rails. The findings explained in the paper lead to the advantage of using the classification method which is relatively simple and can be easily adopted by end users.

2. Fatigue design curves for riveted connections

As pointed out in the previous section, the fatigue life of riveted railway bridges is governed by particular critical structural details since they undergo a much larger number of loading fluctuations and of stress variations with respect to other members. In short and medium span riveted bridges, and in particular in the twinned beam bridge analyzed here, see Fig. 1, short diaphragm riveted connections were found to be the governing fatigue details. As a result, it was deemed necessary both to elicit similar data from relevant literature, such as Bruhwiler et al. [2] and Stadelmann [13], and to compare them with S–N design curves. For riveted details in shear, the fatigue life under constant amplitude loading may be estimated by means of an S–N curve, that reads:

$$N(\Delta\tau)^m = k \qquad (1)$$

where N is the number of stress cycles to fatigue failure; $\Delta\tau$ defines the applied constant amplitude stress range; k is the constant of the detail category; and m represents the design curve slope of the investigated fatigue detail.

Fatigue test results of riveted joints, for which rivet shear failure occurred, are shown in Fig. 2, together with the design curve provided by Eurocode 1993-1-9 [9], Category $C = 100$. The Category $C = 100$ of Eurocode was chosen in accordance with the suggestions of Di Battista et al. [4]. These authors assumed that a riveted shear splice is similar to a splice jointed with non-preloaded bolts. This comparison shows that Eurocode 3-1-9 [9] predictions are conservative. Differently from the use of bolts, the lack of thread roots and thread run-outs in rivets causes a positive influence on the fatigue endurance, because of the absence of stress concentrations. Further positive influences are due to the inherent safety of design curves and to other factors that are not generally taken into explicit account in codes. Therefore, several material properties were experimentally determined in the following sections, in order to justify the favorable fatigue endurance of riveted shear details. The same favorable fatigue endurance was observed also in fatigue shear tests carried out and reported in the following.

3. The case study

The Meschio bridge, a short span riveted flanged railway bridge built in 1918, was taken out of service in 2006. It had been used in the line Mestre-Cormons, which is located in the Northeastern part of Italy. Some of the original drawings of the Meschio bridge are reported in Fig. 3. The mid-span section of one lane is instead shown in Fig. 4(a) together with the plant and lateral view in Fig. 4(b). The net span of the bridge was 12.40 m. The main horizontal structure was made of two couples of twinned–riveted composite flange beams. Wood beams were located between the coupled beams, with a net distance of 565 mm from web to web of a

b

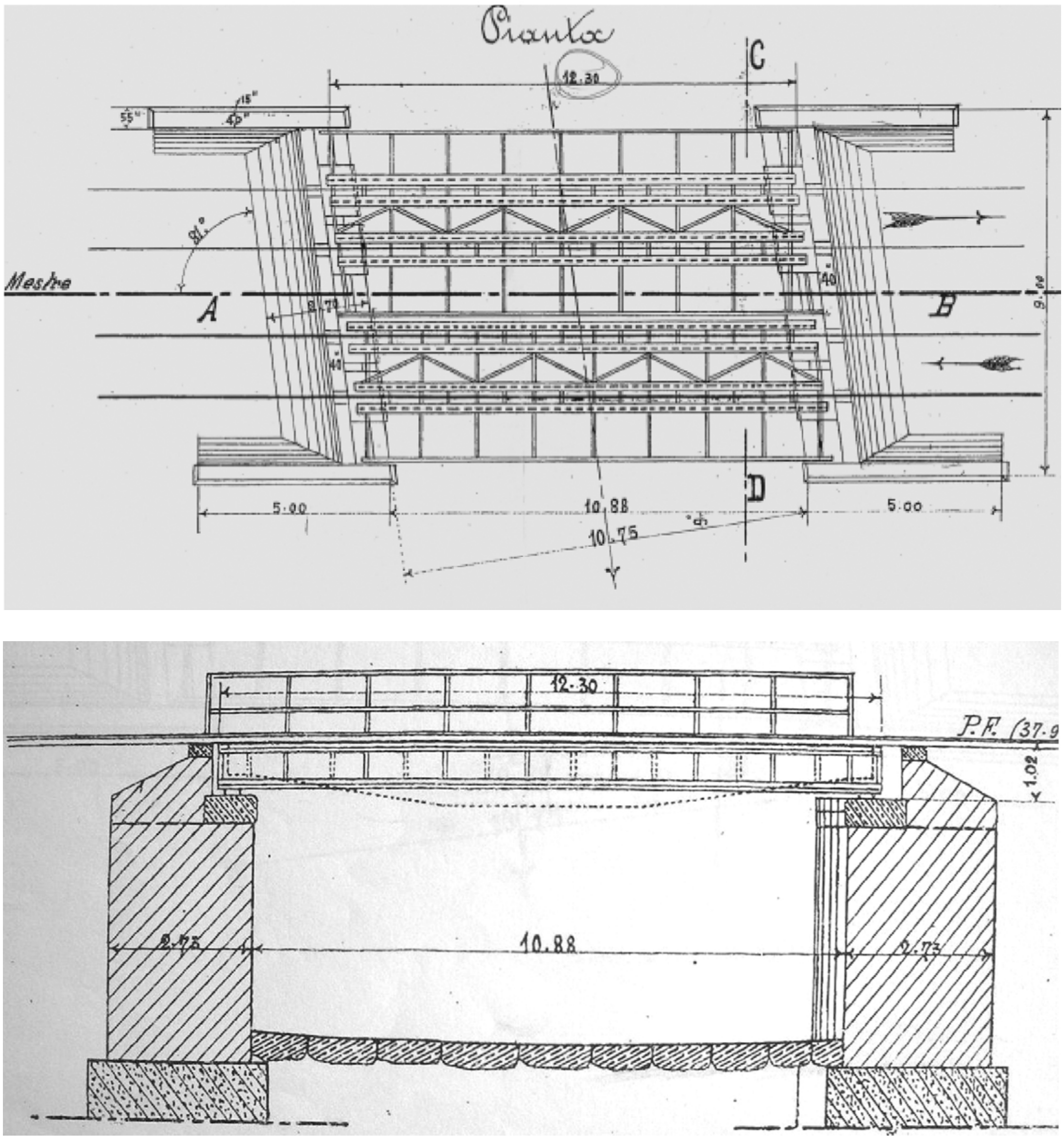

Fig. 3. (a) Original plant; (b) lateral view of the 1918 Meschio bridge.

the beams, while the beam height was 838 mm. In this open-deck riveted railway bridge, transversal short shear diaphragms riveted with double angles to both webs carried the rails, as illustrated in Fig. 4(a). Each twinned beam supported the wood elements of a single rail. The thickness of the main beam plates was 11 mm. While the web was reinforced by 1 m spaced shear stiffeners, the flanges instead were reinforced with 10 mm thick plates. The plate thickness increased from the abutment to the mid-span. Each pair of twinned beams was linked to the corresponding pair with transverse bracing frames, as depicted in Fig. 4(a).

4. Material characterization and full-scale tests

Owing to the presence of notches and the resulting high stress concentration factors, the fatigue behavior of riveted connections is in most cases hardly affected by the material quality and by the mean stress level. Nonetheless, the reduced level of stress concentration as well as the presence of a low ultimate strength, makes the fatigue strength of riveted connections more favorable and sensitive to material quality [14]. Moreover, the development of an accurate inspection strategy related to fatigue assessment requires adequate considerations of the many factors involved, and also a deep knowledge of material properties [2]. As a result, material and mechanical tests for the basic aged materials were carried out as presented in Section 4.1. In order to confirm the favorable comparison with literature data presented in Section 2 also for full-scale details of actual railway bridges, these tests were followed by fatigue shear tests on riveted shear diaphragms as reported in Section 4.2. The main results are summarized herein, while more detailed information can be found in Pipinato [7].

4.1. Material characterization tests

For aged metal bridges that were built between 1870 and 1940 material parameters are in many cases not available. In this respect, a recent European research project, Sustainable bridges [15], suggests that the highest attention must be paid to mechanical properties when dealing with puddle iron and aged steel. In fact, due to the various production processes the iron exhibit a large amount of slag and inclusions along with a great anisotropy. At the same time, aged steels do not usually fulfill the requirements of normalized materials according to EN 10025 [16].

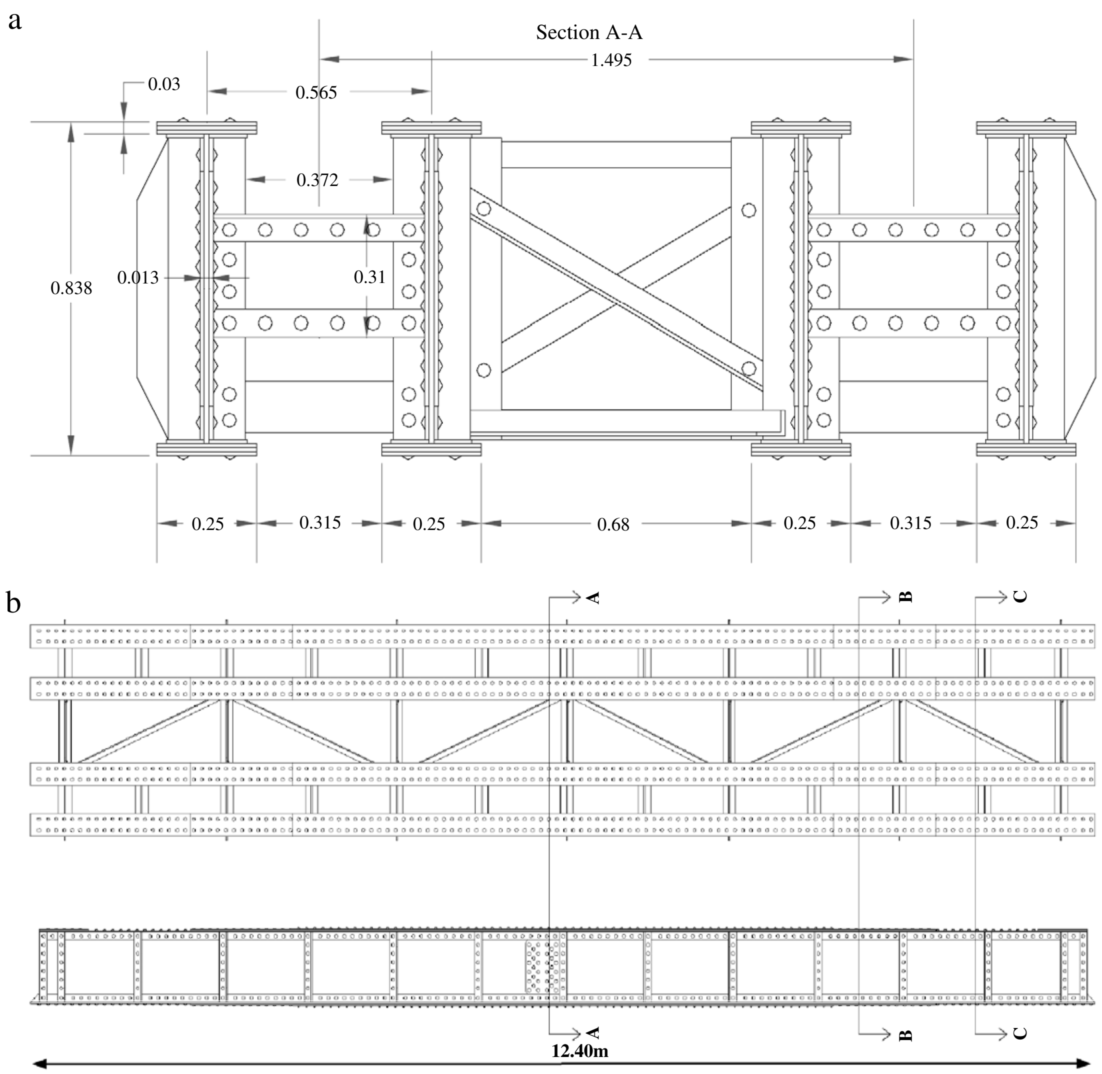

Fig. 4. Twinned girder structure of the Meschio bridge: (a) mid-span section A-A of one lane; (b) plant and lateral view of one lane. Measures in m.

Table 1
Tensile test results.

	Yield strength (MPa)	Tensile strength (MPa)	Elongation A (%)	E (MPa)
Medium value (22 specimens)	322	421	33	221876
COV (22 specimens)	0.0217	0.0208	0.041	0.108

4.1.1. *Tensile tests*

Steel specimens were taken from beam webs and the base material mechanical properties were determined with reference to UNI EN 10002/1 [17]. The constituent materials resulted in having a mechanical strength within the range of S275 and S355 steel (EN 10025 [12]), see Table 1, with a mean yield strength $f_{y,m} = 322$ MPa and a mean ultimate strength $f_{u,m} = 421$ MPa. In agreement with Sustainable bridges [15], these data are not in line with aged steels and iron, but appear to be more consistent with steel produced by a Martin–Siemens process.

4.1.2. *Impact strength tests*

Six specimens with a cross section of 10×10 mm and a *V* notch were tested at ambient temperature, i.e. at 20 °C, with a nominal energy of 300 J. The relevant CVN results are listed in Table 2. The average CVN value of 11.5 J is lower than the reference value of 27 J at 0 °C or −20 °C as suggested in EN10025 [16] for modern steel. Sustainable Bridges [15] suggests similar CVN values and highlights the fact that constitutive materials of old bridges are characterized by the presence of metals that are more brittle than modern metals.

Table 2
CVN test results.

Specimen	Temperature (°C)	Average impact energy (J)
C_1	20	2
C_2	20	10
C_3	20	10
C_4	20	11
C_5	20	11
C_6	20	25
Average value	20	11.5

4.1.3. *Quantometric and metallographic tests*

To perform quantometric and metallographic tests, a Glow Discharge Atomic Emission Spectrometer was employed. The relevant results are collected in Table 3. The micrographic investigation showed a homogeneous grain distribution between ferrite and

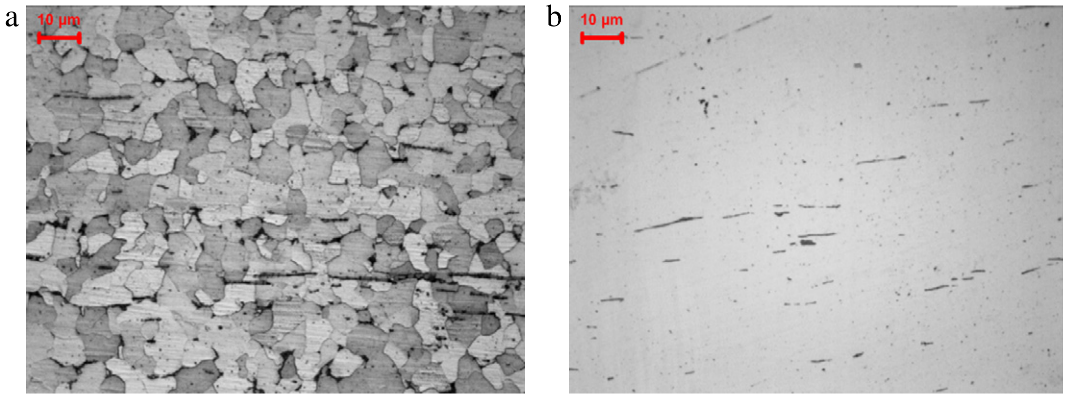

Fig. 5. Metallographic test images: (a) grain structure with chemical treatment; (b) ×200 magnification grain structure without chemical treatment.

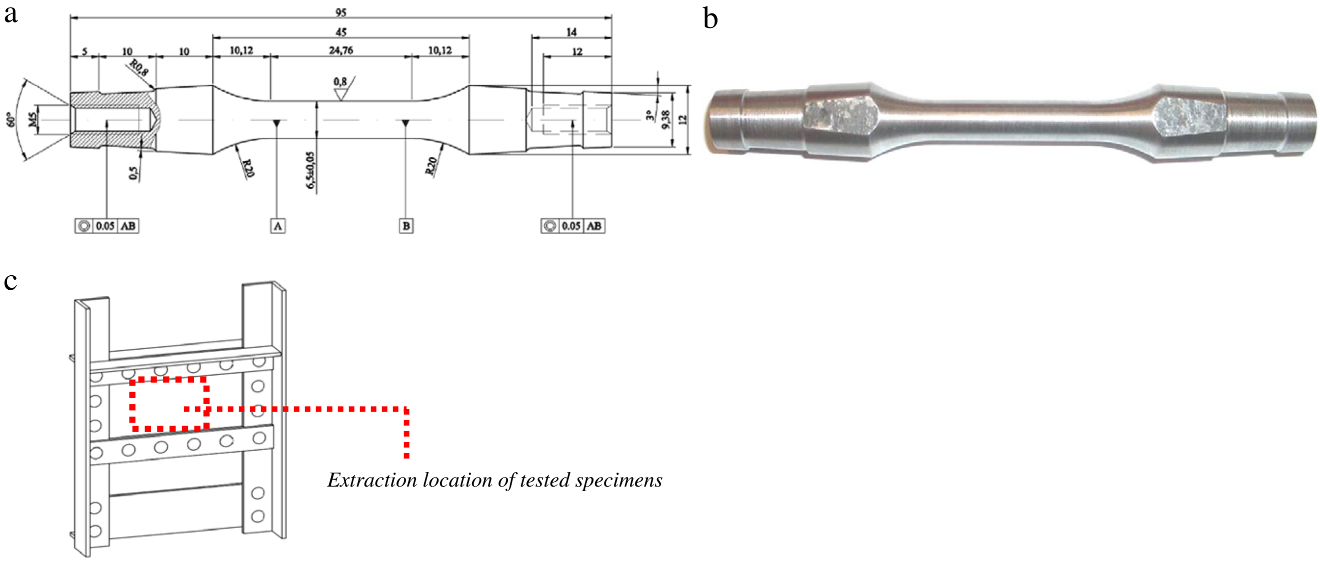

Fig. 6. Rotating bending sample, (a) geometrical scheme; (b) manufactured sample; (c) extraction location of tested specimens. Measures in mm.

Table 3
Quantometric chemical test results.

Component	Spec. 1 (wt%)	Spec. 2 (wt%)
C	0.025	0.060
Mn	0.35	0.490
Cr	0.006	<0.005
Ni	0.02	0.041
Mo	0.0010	–
Co	0.010	<0.040
P	0.036	<0.005
S	0.050	0.029

perlite with 10–30 µm grain size. In Fig. 5 we can observe the presence of the ferrite grain (light) and the perlite (dark) with about a 5% concentration, where a ×200 magnification highlights a sulphur inclusion.

The analysis of the microstructure, performed along with the chemical analysis, leads us to conclude that the constituent material of aged steel exhibited a high sulphur and a low carbon content. With regard to the sulphur content, the micrographic image of a steel not subject to chemical attacks suggests that the steel under exam is not de-sulphurated, while its sulphur content is about twice the maximum value of the quantity commonly present in modern steel (EN10025 [16]). Moreover, the comparison between these chemical results and those published by Mang and Bucak [18], Hohlwegler [19] and Stier et al. [20], confirms that we are dealing with an aged steel, with no precise equivalence to a modern mild steel, except for the tensile strength characteristics. Nonetheless, the high sulphur content has a negative effect on both corrosion resistance and metal toughness, which in turn are two factors also negatively affecting the remaining fatigue life (Höhler [21]).

4.1.4. *Vickers tests on rivets*

The surface hardness of a metal is heavily dependent on its deformation state and consequently on its dislocation density. Therefore four Vickers tests were carried out on rivets and an average value of HV145 with a load of 294 N was achieved. These results correspond to an ultimate strength f_u of about 500 MPa. This value is also in line with the rivet material Fe44B, as suggested by the Technical code for riveting of the Italian Railway Network Authority [22].

4.1.5. *Rotating bending fatigue tests*

Fatigue properties are often correlated with tensile properties by means of rotating bending fatigue tests [23]. Therefore, cylindrical samples were shaped according to ISO1143 [24] as shown in Fig. 6(a),(b). Before carrying out each test, the orientation of the samples was metallographically checked with respect to the rolling direction. Moreover, the specimens were taken from plates in the central location of the short diaphragm as illustrated in Fig. 6(c).

Table 4
Rotating bending fatigue test results and sample dimensions.

Sample	d (m/m)	$\Delta\sigma$ (MPa)	Load (kg)	Number of cycles	Eccentricity (mm)
1	6.51	300	13,960	1,987,700	0.03
2	6.54	330	15,609	594,200	0.02
3	6.52	350	16,708	268,300	0.04
4	6.52	360	17,258	126,200	0.02
5	6.53	380	18,357	62,600	0.03

Note: all specimens exhibit a parallel orientation with respect to the rolling direction.

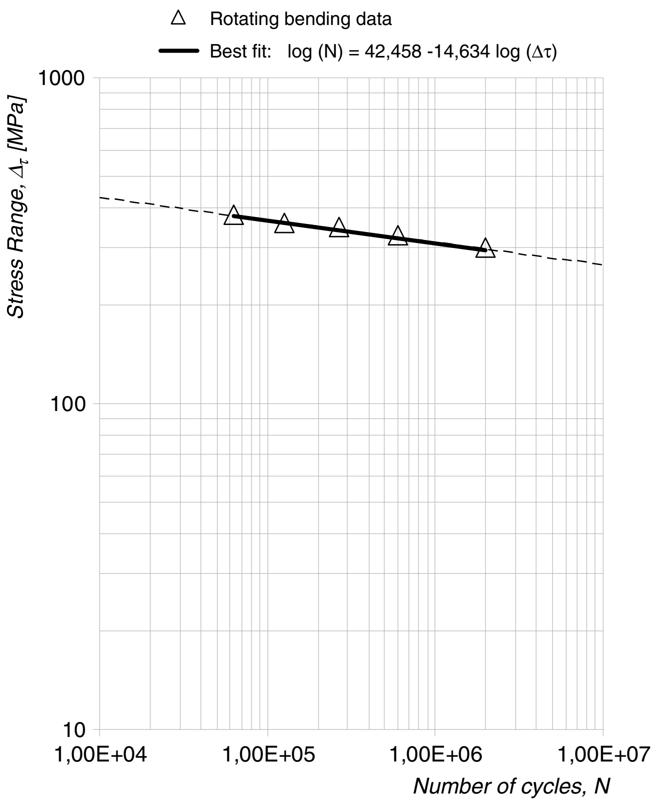

Fig. 7. S–N curve relevant to rotating bending tests.

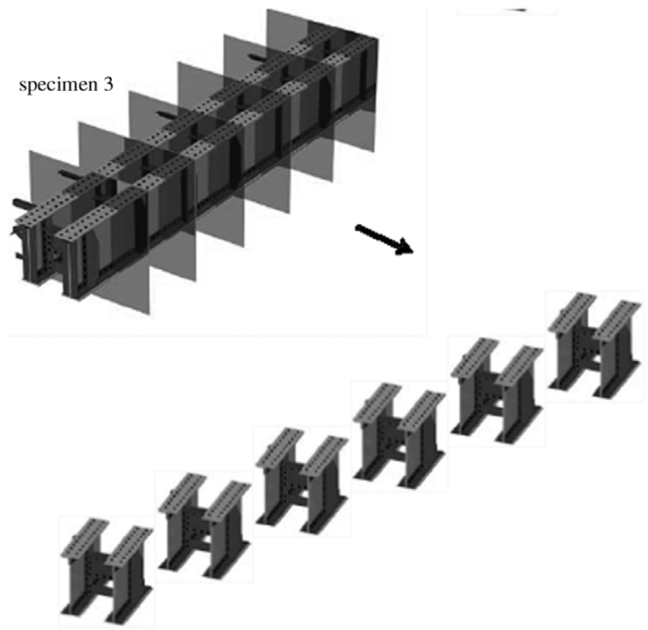

Fig. 8. Dismantling operation of the bridge in order to obtain specimens for both monotonic and high-cycle shear fatigue tests.

The specimens were subjected to different variable stress $\sigma_{AD,N}$ values with a conventional survival up to $N = 10^7$ cycles, a value that was considered a run-out. The relevant results are reported in Table 4 and in Fig. 7. The associated best fit line shown in Fig. 7 is typical of the well-finished metal specimens [23] from which the value $\sigma_{AD,N} = 265$ MPa at $N = 10^7$ can be extrapolated. The relevant fatigue endurance ratio K_N reads

$$K_N = \sigma_{AD,N}/\sigma_R = 0.63 \quad (2)$$

where $\sigma_R = 421$ MPa from Section 4.1.1. In general, the K_N ratio of cast and wrought steel varies between 0.55 (for carbon mild steel with $f_u = 400$ MPa) and 0.35 (for high strength steel with $f_u = 1600$ MPa [23]). As a result, the tested material exhibits a fatigue endurance ratio in line with modern mild steel.

4.2. Shear tests on short diaphragm connections

The critical members most subjected to fatigue in railway bridges are generally characterized by a short influence length. Therefore, they undergo a much larger number of loading fluctuations and of stress variations with respect to other members. For this reason, the fatigue life of a riveted bridge will be also governed by the performance of these details.

By means of additional tests, Bruhwiler et al. [2] highlighted that the tensile stress range in bottom flanges of main beams was lower than that recorded in the rivets of short diaphragms. Therefore these riveted connections were subject to both more cycles and to higher stress variations than other parts of the bridge. As a result, the assessment of riveted connections, and in particular of composite built-up elements to shear failure, becomes an essential step of a typical riveted bridge evaluation. Relevant tests and results are reported herein.

4.2.1. Monotonic test

In order to understand the actual mechanism of failure, a monotonic test in simple shear was performed on the shear riveted connections of a single short diaphragm (see Figs. 8 and 9(a)).

The test was carried out up to the failure of the local riveted connections in the stiffener-to-web part. As shown in Fig. 9, an abrupt failure occurred at a load level of 1060 kN, approximately corresponding to the shear stress value on single rivets of the 2nd and 3rd rows of 470 MPa.

4.2.2. High-cycle fatigue tests

In order to characterize the detail category and to evaluate the benefit of the bridge material properties on fatigue resistance, shear tests on short diaphragm riveted connections that carried the rails were performed.

During the tests, each specimen was bolted to a hydraulic actuator as shown in Fig. 10. Four specimens were tested with different load ranges; in detail, specimen I was loaded with 184,000 cycles up to failure, while specimens II, III and IV were tested up to 560,000, 208,850 and 504,515 cycles, respectively. All specimens were loaded with a 5 Hz frequency, with stress levels in the elastic phase. Microcracks, induced by tensile stresses, did appear both in

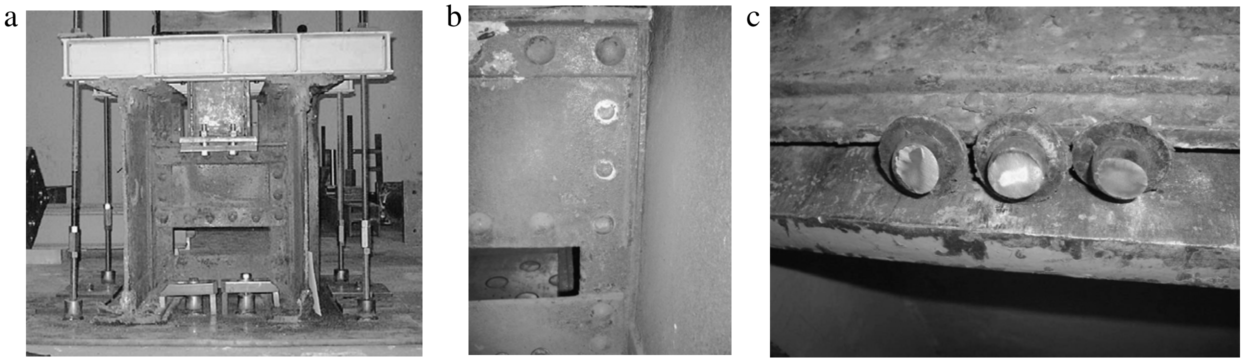

Fig. 9. Monotonic shear test on a short diaphragm riveted connection: (a) test setup; (b) loosing rivet on the diaphragm connection; (c) failed rivets.

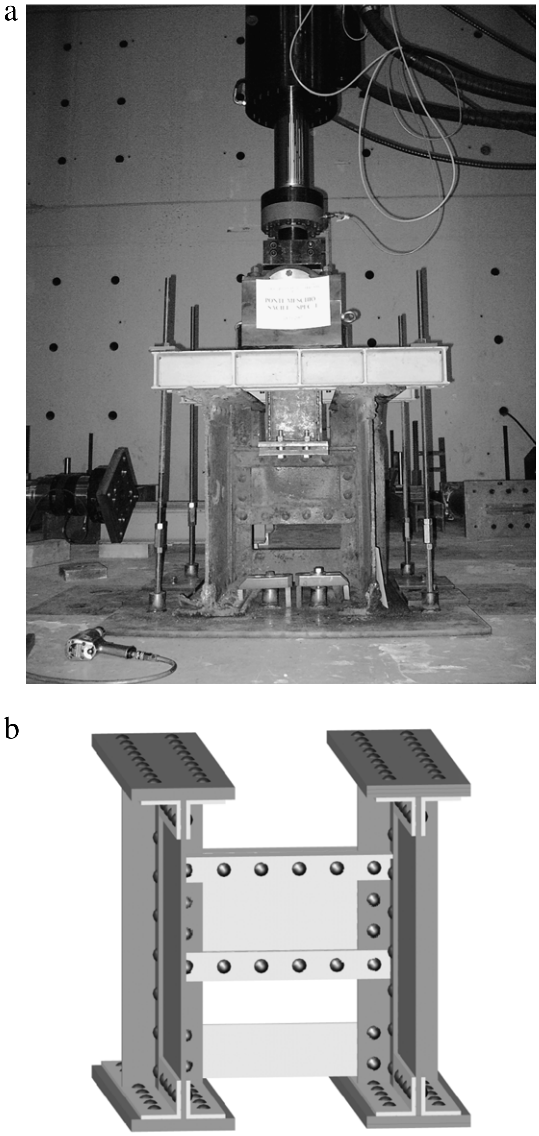

Fig. 10. High-cycle fatigue test: (a) test setup; (b) single specimen.

the central plates and in the angles and in the lateral webs of each specimen, as reported in Table 5. In detail, a crack arising from the rivet shank had developed from the upper connected plate, so that the rivet head fell off the plates, see Fig. 11, and the clamping effect was significantly reduced. This type of crack failure is generally difficult to detect because no external sign of propagation appears.

Fig. 11. View of the shank section of one failed rivet.

After failure, only an external slippage of the rivet shank of about 1 mm can be observed; however, no rivet failure and no fatigue cracking had been detected in the connections of the bridge during its past service life.

The number of cycles listed in Table 5 does not take into account the in-service cycles performed during the previous life of the bridge. Therefore, the forthcoming evaluations are on the safe side. The test results are collected in the logarithmic Wohler diagram illustrated in Fig. 12. The fatigue design curve of EN 1993-1-9 [9], Category 100, is compared both with experimental values and with the corresponding best fit line. The Category $C = 100$ provided in Table 8.1 of EN 1993-1-9 [9] is the only available category for bolted connections in shear. Though this code does not provide specific fatigue design curves for riveted connections, the Category $C = 100$ is on the safe side. This trend confirms: (i) the validity of the comparison presented in Section 2 about the safety level of EN 1993-1-9 [9] regarding this detail; (ii) the conclusion of Di Battista et al. [4] about the equivalence between riveted shear splices and splices jointed with non-preloaded bolts; (iii) the positive effects of the aged material properties of the bridge on fatigue endurance.

5. Conclusions

In order to study the high-cycle shear fatigue behavior of metal riveted railway bridges, an experimental investigation was performed on a 12.4 m span railway bridge taken out of service in Italy in 2006. Initially, a full characterization of materials was carried out and a yield strength f_y of about 322 MPa and a tensile strength f_u of about 421 MPa were determined. Microstructure analysis highlighted the typical configuration of an aged steel material. Along with it a CVN of about 11.5 J, typical of constitutive materials of aged bridges was determined, and it was also found out via rotating bending tests that the aged steel exhibited a

Table 5
Results of shear fatigue tests.

Specimen	Maximum load (kN)	Minimum load (kN)	Stress range $\Delta\tau$ (MPa)	N. of cycles	Remarks
I	900	100	352.9	138,000	Paint cracks under the loaded rivets
				150,000	Paint cracks under the rivets loaded in the rear part
				159,000	Visible rivet head deformation
				176,000	2–3 mm detachment rivet heads from the central plate in the rear left part
				184,000	Failure: front side, right rivets/2nd–3rd rows
II	800	100	308.77	23,300	Paint cracks under the loaded rivets
				188,400	1–2 mm detachment rivet heads from the central plate in the rear left part
				560,000	Failure: rear side, right rivet/3rd row
III	850	100	330.82	203,850	Paint cracks under the rivets loaded in the rear part
				208,850	Failure: rear side, right rivet/3rd row
IV	700	100	264.66	430,515	Paint cracks under the rivets loaded in the rear part
				504,515	Failure: rear side, right rivet/3rd row

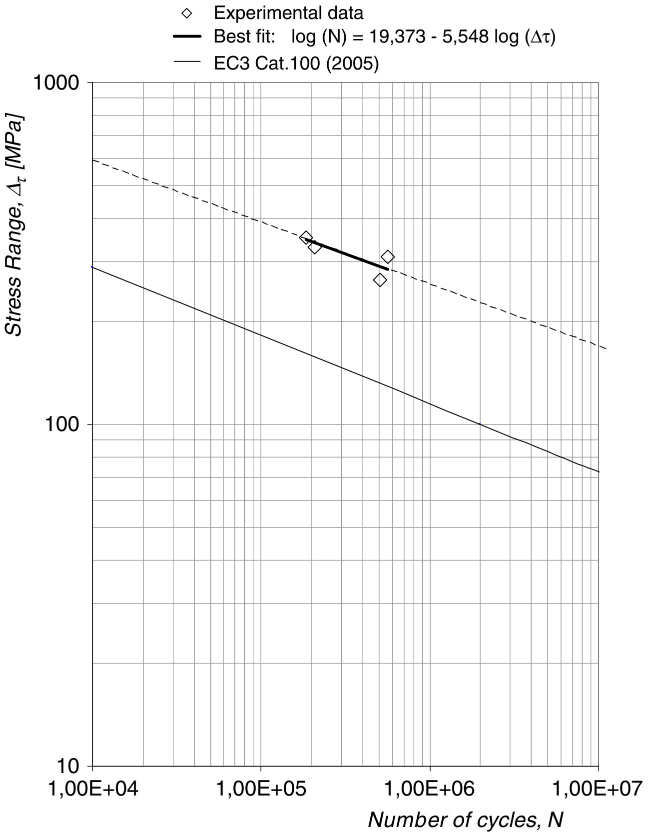

Fig. 12. Test data, best fit and fatigue design curve Category 100 from Eurocode 3-1-9 (2005).

positive effect on fatigue endurance with an endurance ratio K_N of 0.63. All the analysis performed on the base material, lead to the conclusion that the aged material exhibited mechanical values in line with modern mild steel, but is more fragile and had a different microstructure configuration. Then, due to the short influence length of the short shear diaphragms with respect to other members, the hot-spot details of this bridge were the riveted connections of the short shear diaphragms that carried the rails. Consequently, all monotonic and high-cycle fatigue experimental tests focused on shear riveted connections, in order to allow relevant failure mechanisms to be understood. The literature data and tests were compared to EN 1993-1-9 [9] design curves. It was shown that the shear Category 100 of EN 1993-1-9 [9] is on the safe side and that the equivalence between riveted shear splices and splices jointed with non-preloaded bolts as suggested by Di Battista et al. [4] is also adequate for the European code.

These results for riveted historical metal bridges highlight that a shear failure mode, originated from the rivet shanks of short diaphragms, has to be considered accurately and inspected timely. Finally, an overall assessment of hot-spot details requires further studies with probabilistic approaches for both loading and fatigue strength.

Acknowledgments

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References

[1] ASCE. Committee on fatigue and fracture reliability of the committee on structural safety and reliability of the structural division. Fatigue reliability 1–4. Journal of Structural Division. Proceedings of ASCE 1982;108(ST1):3–88.
[2] Bruhwiler E, Smith IFC, Hirt M. Fatigue and fracture of riveted bridge members. Journal of the Structural Engineering 1990;116(1):198–213.
[3] Kulak GL. Discussion of fatigue strength of riveted bridge members by J.W. Fisher, B.T. Yen, D. Wang. Journal of the Structural Engineering, ASCE 1992; 118(8).
[4] Di Battista JD, Adamson DE, Kulak GL. Fatigue strength of riveted connections. Journal of the Structural Engineering 1997;124(7):792–7.
[5] Bursi OS, Ferrario F, Fontanari V. Non-linear analysis of the low-cycle fracture behavior of isolated Tee stub connections. Computers & Structures 2002; 80(27–30):2333–60.
[6] Matar EB, Greiner R. Fatigue test for a riveted steel railway bridge in salzburg. Structural Engineering International 2006;16(3):252–60.
[7] Pipinato A. High-cycle fatigue behavior of historical metal riveted railway bridges. Ph.D. thesis. Italy: University of Padova; 2008.
[8] Pipinato A, Molinari M, Pellegrino C, Bursi O, Modena C. Fatigue tests on riveted steel elements taken from a railway bridge. In: Structure and infrastructure engineering. Taylor and Francis; 2009 [in press].
[9] EN 1993-1-9. Eurocode 3: Design of steel structures—part 1–9: Fatigue, CEN, Brussels. 2005.
[10] Kühn B, Luki M, Nussbaumer A, Günther H-P, Helmerich R, Herion S, et al. Assessment of existing steel structures: Recommendations for estimation of remaining fatigue life—Joint Report Prepared under the JRC–ECCS cooperation agreement for the evolution of Eurocode 3 (programme of CEN/TC 250). In: Sedlacek G, Bijlaard F, Géradin M, Pinto A, Dimora S, editors. 1st ed. Background documents in support to the implementation, harmonization and further development of the Eurocodes, 2008. February 2008, EUR 23252 EN.
[11] Al-Emrani M, Kliger R. FE analysis of stringer-to-floor-beam connections in riveted railway bridges. Journal of Constructional Steel Research 2003;59: 803–18.
[12] Brencich A, Gambarotta L. Assessment procedure and rehabilitation of riveted railway girders. The Campasso Bridge Engineering Structures 2009;31(1): 224–39.
[13] Stadelmann W. Compiling of material tests on wrought iron rivets at EMPA, St. Gallen. Private communication, cited in [2]; 1984 [in German].
[14] Gurney TR. Fatigue of welded structures, Cambridge, 1968.
[15] Sustainable bridges. Guideline for Load and Resistance Assessment of Existing European Railway Bridges—Advices on the use of advanced methods. European research project under the EU 6th framework programme. http://www.sustainablebridges.net/; 2006.
[16] EN 10025. Hot rolled products of non-alloy structural steels. Technical delivery conditions for flat products, ISBN 058044779 0, Committee Ref. ISE 12, 2004.
[17] UNI EN 10002/1. Metallic materials – Tensile testing – Part 1: Method of test at ambient temperature. UNI, Milan; 2004.

[18] Mang F, Bucak Ö. Tauglichkeit und Lebensdauer von bestehenden Stahlbauwerken, Kapitel 18 in Stahlbau Handbuch 1, Teil B; Stahlbau–Ein Handbuch für Studium und Praxis, 3 Bände. Herausgegeben vom Deutschen Stahlbau-Verband–Köln. Stahlbau-Verlagsgesellschaft mbH Köln; 1996.
[19] Hohlwegler B. Korrosion von Stählen aus eisernen Konstruktionen des 19. Jahrhunderts, Mineralogische und metallographische Untersuchungen an historischen Eisen und an daraus resultierenden Korrosionsprodukten. Dissertation Universität Karlsruhe (TH); 1993.
[20] Stier W, Kosteas D, Graf U. Ermüdungsverhalten von Brücken aus Schweißeisens. Der Stahlbau 5; 1983.
[21] Höhler S. Material properties of Metal Railway Bridges, Technical report, Sustainable Bridges. WP4-S-R- 001; 2005 [see also Sustainable bridges 2006].
[22] Italian Railway Network Authority-RFI. Instruction 44/t, Specifica tecnica relativa al collaudo dei materiali e alla costruzione delle strutture metalliche per ponti ferroviari e cavalcaferrovia, Technical code of the Italian Railway Network Authority; 2000 [in Italian].
[23] Andreini P. Manual of Engineering Mechanics. Hoeply; 1994 [in Italian].
[24] ISO1143. Rotating bar bending fatigue testing, International organization for standardization technical committee ISO/TC 164/SC 5; 1975.