Proceedings of the Eleventh International Conference
on Computational Structures Technology,
B.H.V. Topping, (Editor),
Civil-Comp Press, Stirlingshire, Scotland

Paper 107

Experimental Analysis and Fatigue Assessment of a Railway Steel Viaduct

L. Vincenzi[1], M. Savoia[2] and W. Salvatore[3]
[1]DIMEC, University of Modena and Reggio Emilia, Italy
[2]DICAM, Structural Engineering, University of Bologna, Italy
[3]Consorzio Pisa Ricerche, Italy

Abstract

In this paper, the preliminary results concerning the fatigue assessment of the Lagoscuro railway viaduct are described. Experimental dynamic tests have been performed in order to identify the modal properties of the bridge. Then, a finite element model is corrected with a model updating procedures, where the uncertain model properties are adjusted in order to have the numerical predictions as close as possible to the measured data. Making use of the finite element model, a preliminary fatigue assessment is carried out following the procedure defined in EN1993.

Keywords: fatigue analysis, dynamic identification, railway, steel viaduct, riveted joint, model updating, evolutionary algorithm, local vibrations.

1 Introduction

The functionality maintenance of infrastructures like bridges is acquiring more and more importance due to the huge economic losses related to the interruption of their regular service. In particular, fatigue represents one of the more diffused failure modes in steel and composite steel-concrete bridges [1-3]. Several studies were performed in the past in order to assess the fatigue resistance of steel and steel-concrete composite bridges; such studies were the base of modern codes and standards. Despite of these efforts, the fatigue assessment of railway bridges, for both design of new bridges and assessment of existing ones, is one of the main issues in current practice. In fact, phenomena like "vibration induced" and "distortion induced" fatigue are still partially uncovered by the actual design codes and represent critical aspects for the assessment of existing bridge remaining life and for the design of new bridges.

In this paper, some results concerning the fatigue assessment of Lagoscuro railway viaduct are described. The Lagoscuro viaduct is composed of 2 parallel steel railway viaducts crossing the Po river. The first viaduct was built in 1948 and is composed by 9 single span truss-girder bridges; the upper and the lower chord, diagonals and stunts of the main truss girders are composed of 4 L-shaped steel elements, riveted together by means of plates; stringers and additional elements supporting the railway lines are also riveted. The new Lagoscuro viaduct was recently built in order to potentiate the railway line; the same geometry has been adopted but, differently to the old viaduct, the truss girders are composed by H-shaped elements welded or bolted together in the joints.

In the present paper, results of experimental dynamic tests [4-5], performed in order to identify the modal properties of both viaducts, are first presented. Then, global FE models are corrected according to the model updating procedures, where the uncertain model properties are adjusted in order to obtain numerical predictions as close as possible to the measured data, in term of modal frequencies and mode shapes [6]. An evolutionary algorithm [7-8] is used and the sensitivity of identification parameters is investigated.

Global models are used to perform a preliminary fatigue analysis. The fatigue assessment is carried out following the procedure defined in the section 6 of EN1991 – Eurocode 1 [10]. At the same time, the global models are used to find if and where local vibrations may occur in structural members. After having identified the members potentially subject to fatigue or local vibrations, a substructure finite element model containing the specific member has been studied. The detailed substructure model has been then excited by prescribing at the boundaries the displacement time histories obtained in the corresponding nodes of the global model. A linear dynamic transient analysis has then been performed. From the stresses obtained in the detailed FE model, the fatigue assessment is carried out following the procedure defined in EN1993 – Eurocode 3 (part 1-9) [9] and the results are compared to those obtained with the procedure proposed in [4]. In order to obtain the stress ranges on each element, dynamic analyses are performed with traffic mixes defined according to EN 1991 - Eurocode 1 [10]. Stress range spectra are then determined and the damage indexes are evaluated according to the Miner role.

2 The case study

The Lagoscuro viaduct is a steel viaduct on the Bologna –Venice railway line. It is composed by 2 parallel steel railway viaducts crossing the Po river (Figure 1).

The first viaduct was built in 1948 and is composed by 9 single span truss-girder bridges. The 5 inner spans are 75 meters long, while the 4 end spans are about 60 meters long. Two main truss girders support the vertical load, while X-shaped upper and lower lateral bracing assure the transverse stability. The upper and the lower chord, the diagonals and the stunts are composed of 4 L-shaped steel elements, riveted together by means of plates; stiffening plates are also designed in order to improve the transverse stiffness of the cross-section. Stringers and additional elements supporting the railway lines are also composed of riveted steel elements. End supports are constituted by steel bearings directly in contact with lower main strings, so that a simply supported scheme is realized.

The new Lagoscuro viaduct was recently built (2005) in order to potentiate the railway line; the same geometry is adopted but, differently to the old viaduct, the

Figure 1. The Lagoscuro viaduct on the Po river (old viaduct on the left, new viaduct on the right).

truss girders are composed by H-shaped elements bolted together in the joints. From a preliminary fatigue analysis, truss girder elements of the new Lagoscuro viaduct are correctly designed against fatigue. Moreover, as far as the local vibrations of bracing are concerned, thicker sections are adopted: higher vibration frequencies and, consequently, reduced fatigue effects are expected. For this reasons, in the following only the results for the old Lagoscuro riveted bridge will be described. Since the same details for the connections have been adopted, similar results are obtained for the 75 meters long spans and the 60 meters long spans of the bridge. Hence, for the sake of brevity, this paper focuses on the side shortest spans only.

3 Dynamic experimental analysis and modal updating

A finite element model of the bridge, indicated as “global model” in the following, has been built and calibrated using experimental dynamic data (firsts 4 natural frequencies and mode shapes). The dynamic experimental campaign is summarized in Section 3.1. The global model is then described in Section 3.2 and the updating procedure is illustrated in Section 3.3. The local models and the fatigue assessment analysis are finally reported in Section 4.

3.1 Dynamic experimental campaign

The accelerations in the vertical and (transverse) horizontal directions of suitable key points of the structure were measured by using PCB piezoelectric accelerometers with a sensitivity of 10 Volt/g. They have been connected to a LMS data acquisition system for data recording and processing. The sampling frequency adopted was 400 Hz for time periods of 20 minutes at least. The ambient vibrations as excitation source has been used, whereas those due to the train transit during the acquisition of ambient vibrations were not considered.

The eleven accelerometers were placed in a number of selected positions on the bridge deck: 6 instruments to measure vertical accelerations (two at mid-span, four at quarters, on both sides of the deck), 3 for the horizontal accelerations (at mid-span

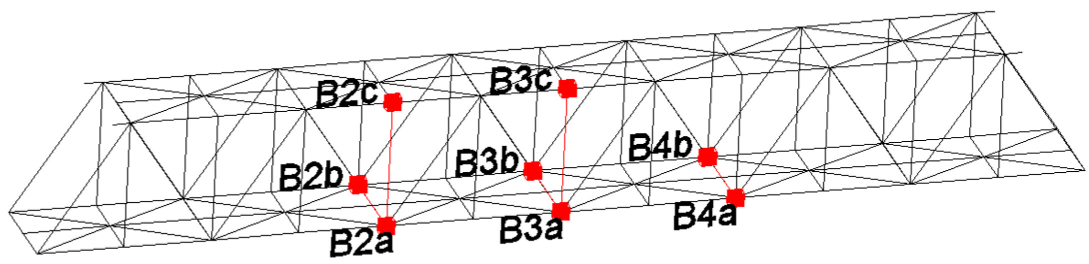


Figure 2. Layout of the bridge with the instrument positions.

and at quarters, in the lower chord) and, finally, 2 for horizontal accelerations at mid-span and at quarter, in the upper chord. The scheme of the instrument positions is shown in Figure 2.

Starting from the recorded data, the global modes are identified by Operative Modal Analysis technique (OMA) [11-12], using the poly-reference Least Squares Complex Frequency domain (p-LSCF) algorithm, also known under its commercial name PolyMAX [13-14]. Ten global modes of vibration were clearly identified from the experimental tests. The first frequencies and mode shapes extracted from the identification procedure are reported in Table 1, together with the corresponding mode shapes obtained from the FE model.

The old Lagoscuro bridge was designed mainly to support vertical loads. Due to the presence of the vertical truss girders, the bridge has a high stiffness in the vertical plane but is quite deformable in the transverse direction. Therefore, the first mode shape (2.14 Hz) is characterized by the horizontal deformation of the bridge, with a significant cross-section distortion. The second mode shape (2.85 Hz) is a flexural mode with deflection in the vertical plane. The third mode shape is a torsional mode and the forth is the second lateral.

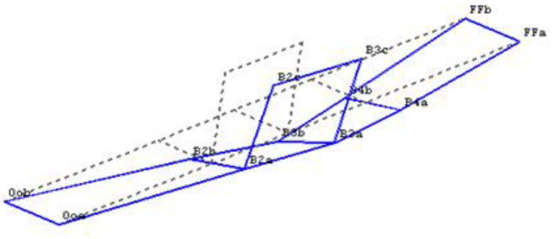
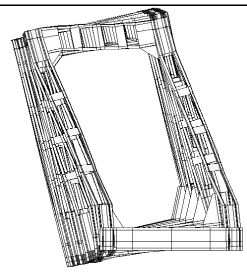
3.2 The global model

The dynamic behaviour of the old riveted bridge is studied with a global Finite Element model developed by ANSYS software [15]. Linear elastic shell elements are used for the two main steel truss girders, stunts and floor-beams, and beam elements for the upper and lower bracings. As far as the external supports are concerned, the displacements in the vertical direction are set to zero, whereas translational springs are introduced to model the stiffnesses in the horizontal directions.

3.3 The updating procedure

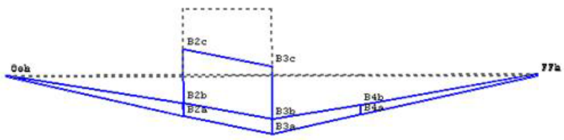
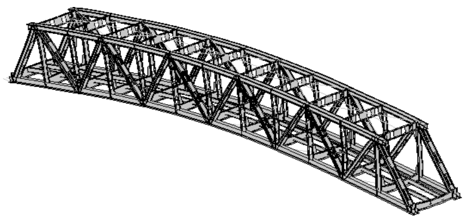
The global Finite Element model is used to reproduce the global dynamic behaviour of the bridge. For this purpose, the FE model is corrected according to a model updating procedures. The model updating procedure is based on an optimization problem, where the goal is to obtain the optimal values of system parameters minimizing an objective function, written as a function of the distance between

Mode shape n. 1: I Lateral

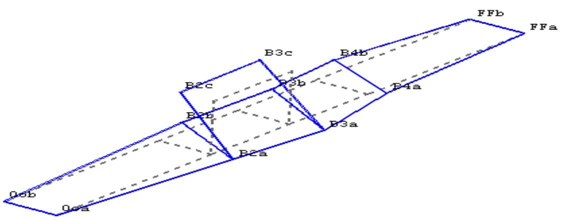
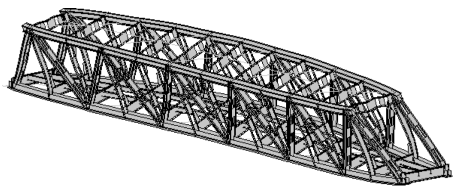
Frequency 2.143 Hz

Mode shape n. 2: I Flexural

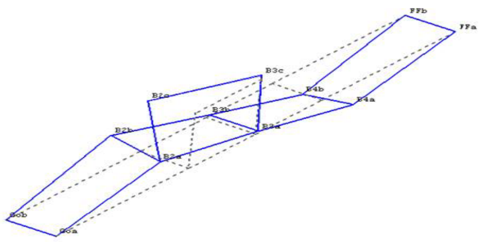
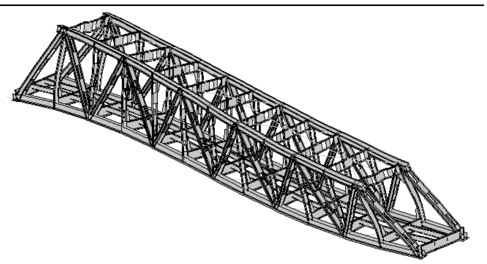
Frequency 2.857 Hz

Mode shape n. 3: I Torsional

Frequency 4.307 Hz

Mode shape n. 4: II Lateral

Frequency 4.700 Hz

Table 1. The old Lagoscuro Bridge: the first 4 experimental frequencies and mode shapes.

modal parameters obtained from experimental tests and those given by a numerical model of the structure [6].

The objective function is often non-smooth or even discontinuous and can also contain some local minima. The success of the application of the updating method depends on the definition of the optimization problem and the mathematical capabilities of the optimization algorithm [6]. Conventional gradient-based methods (as Newton and Quasi-Newton Algorithms) have an efficient convergence rate, but they may reach local minima depending on the selected starting point. In these algorithms, the local curvatures of the original function are calculated and used to build an approximate quadratic model function. Hence, gradient-base methods often fail or are not accurate due to the ill conditioning of the optimisation problem when the objective function is flat close to the solution. In the case of a number of identification parameters greater than 2-3, global optimization techniques must be employed. Among them, Genetic algorithms and Evolution approaches are considered very effective numerical methods.

In order to obtain the unknown parameters concerning the numerical model of the Lagoscuro old bridge, the optimization problem is solved by a global search method called DE-Q [7-8]. In this algorithm, the response surface methodology [16-17] is introduced in the classical Differential Evolution algorithm to improve its performances. Differential evolution (DE) [18] is an evolutionary algorithm where N different vectors collecting the parameters of the system are chosen randomly or by adding weighted differences between vectors obtained from two populations. In the modified algorithm, the new parameter vector is defined as the minimum of a second-order polynomial surface, approximating the cost function. The performances in term of speed rate are strongly improved by introducing the second-order approximation; nevertheless, robustness of DE algorithm for global minimum search of cost function is preserved, since multiple search points are used simultaneously. More details about the updating procedure can be found in [7-8].

Experimental frequencies and mode shapes are used as the reference for the model. The selected identification parameters are:

- the equivalent density of steel members m_{eq} (due to the large amount of plates and rivets);
- the additional masses on the deck m (due to the presence of some secondary elements and masses that are not modelled);
- the translational (longitudinal) spring stiffness k_h (introduced to simulate the behavior of supports).

The numerical tests are performed by adopting, as the input data, experimental frequencies and mode shape vectors. First, the experimental and the numerical modes are coupled by using the *MAC* (Modal Assurance Criterion) [5]. Then, the objective function to be minimized during the identification procedure is defined as the relative error between modal frequencies and mode shapes obtained adopting a given set of identification parameters $(\omega_i, \boldsymbol{\varphi}_i)$ and the reference solution $(\overline{\omega}_i, \overline{\boldsymbol{\varphi}}_i)$, i.e.:

$$H = \sum_{i=1}^{N} \left[w_1 \left(\frac{\omega_i - \overline{\omega}_i}{\overline{\omega}_i} \right)^2 + w_2 \, NMD_i^2 \right], \tag{1}$$

where NMD is the so called "Normalized Modal Difference" [4,19] defined as:

$$NMD_i = \sqrt{\frac{1 - MAC(\boldsymbol{\varphi}_i, \overline{\boldsymbol{\varphi}}_i)}{MAC(\boldsymbol{\varphi}_i, \overline{\boldsymbol{\varphi}}_i)}} \tag{2}$$

Mode	**Experimental frequency** [Hz]	**Numerical frequency** [Hz]	**Error** [%]	**MAC** [%]
1	2.143	2.166	1.07	96
2	3.857	3.638	5.60	96
3	4.307	4.409	2.37	75
4	4.700	4.749	1.04	88

Table 2. Comparison between experimental and numerical results after the optimization process.

In Eq. (1), $N = 4$ mode shapes are considered. Moreover, the weight constants are taken as $w_1 = 1$, $w_2 = 0.1$.

After the updating process, the numerical frequencies of the numerical model were very close to the experimental values, with errors never greater than 6%, see Table 2. To compare the experimental and numerical mode shapes, the value of MAC is also given in the table.

4 Dynamic analysis on by means of local models

The train transits on the bridge are simulated in order to detect the presence of local vibrations in structural members. In the dynamic analysis of a railway bridge, moving loads are traditionally represented as a series of moving axle loads. This approach is adopted by many design codes, see for instance the Eurocode 1 [10]. In this model, the global dynamic behaviour of the bridge caused by the moving action of the vehicle can be described with a sufficient degree of accuracy. However, the effect of interaction between the bridge and the moving load is neglected. For this reason, the moving load model gives good results only if the mass of the vehicle is small relative to that of the bridge, or when the vibration frequencies of the bridge are well separated to those of the vehicle [20].

Following the procedure described in [21], a 3-dimensional Vehicle Bridge Interaction model is apply to the global model to obtain the displacements in both vertical and horizontal directions. Details of the results are reported in [22]. The masses of cars are defined according to the Eurocode [10] and the fatigue assessment is carried out on the basis of the standard traffic mixes traffic defined in the Annex “D” of EN1991 – Eurocode 1. A preliminary global fatigue analysis has been performed and reported in [23]. The main lower bracing system (see Figure 3) is identified as the one more prone to fatigue and local vibrations. Substructures containing the specific member are then studied. Moreover, the connections between floor-beams and stringers are identified as possible critical details for fatigue. Local models developed to better investigate the connections behaviour are described in the next section.

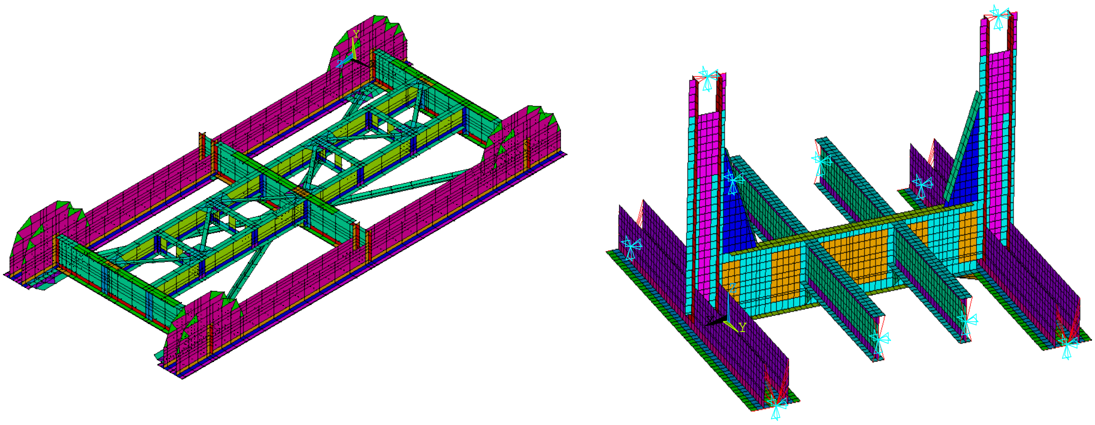
4.1 The local models

The main lower bracing system (see Figure 3) is identified as the one more prone to local vibrations. This system is composed by an X bracing, spanning the whole width of the bridge in one direction and two panels in the other direction. Each bracing is composed by two L-shaped elements connected together by stiffening plates.

The identified critical members and details for fatigue are the floor-beams, intersections between floor-beams and stringers and the intersection between floor-beams and the main lateral truss girders. All the other details seem not to be critical for fatigue.

Two local detailed substructures are then modelled (see Figure 4*a,b*). In all the models, the presence of rivets is modelled with rigid links connecting elements or it is assumed they guarantee the structural continuity.

Figure 3. Details of lower bracings, floor-beams and stringers.



(*a*) (*b*)

Figure 4. Local mode (*a*) for stringers and bracings and (*b*) for floor-beams.

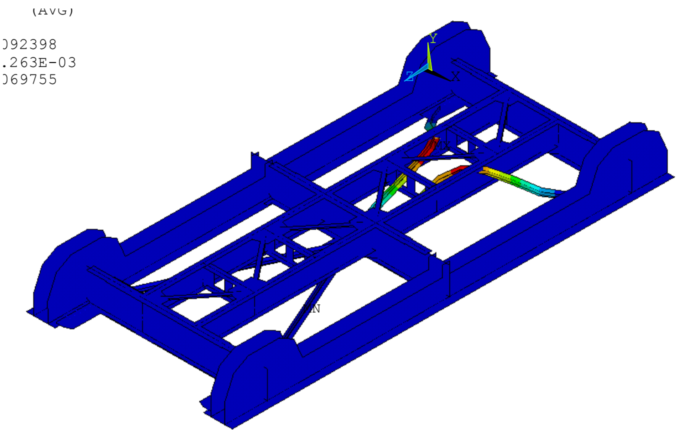


Figure 5. First mode shape of bracing vibrations at 5.6 Hz.

Substructure "*a*" models the bracing system and it is able to gives accurate results for the local vibrations of X bracing elements and stringers. The model is composed by 4600 nodes, 4400 shell elements and 136 links (see Figure 4*a*). In all bridges, only two type of bracings are present. For the local analysis, the most recurrent one was chosen. The main natural modes are obtained by modal analysis and the first bending mode shape (5.6 Hz) of bracings is shown in Figure 5. In the further analyses, the higher modes were neglected due to their very low vibration amplitudes and high frequency.

The model "*b*" aims to faithfully reproduce the connections between floor-beams and stringers, and floor-beams with the two main truss girder. From this model, information about stress history of floor-beams will be obtained. All floor-beams have the same section; the local model is then representative for the whole old viaduct. In the model "*b*", about 5000 nodes, 5000 shell elements and 200 links are used.

Each substructure was subject to a linear transient dynamic analysis, prescribing at the boundaries the same displacement histories obtained from the global dynamic analysis. Examples of time history of stresses in elements caused by the transit of a single train is shown in Figure 6 – 10. The maximum stress for truss girder members attains about 25 MPa in the case of transit of a passengers train (see Figure 6a), 18 MPa for high speed trains (Figure 6b) and 65 MPa for freight trains (Figure 6c). Very high stress are obtained for stringers (see Figure 7): the maximum stress range is greater than 180 MPa when the train type 6 transits on the bridge. As far as the bracings are concerned (Figure 9), the stress history is clearly due to the superposition of low frequency excitation, due to the quasi-static displacements, and higher frequency stresses probably caused by the local vibrations of the member.

5 Fatigue assessment

The fatigue strength for nominal stress ranges is usually represented by a series of (log $\Delta\sigma_R$) – (log N) curves (called S-N lines), which correspond to typical detail categories. The category is designated by a number which represents, in N/mm^2, the

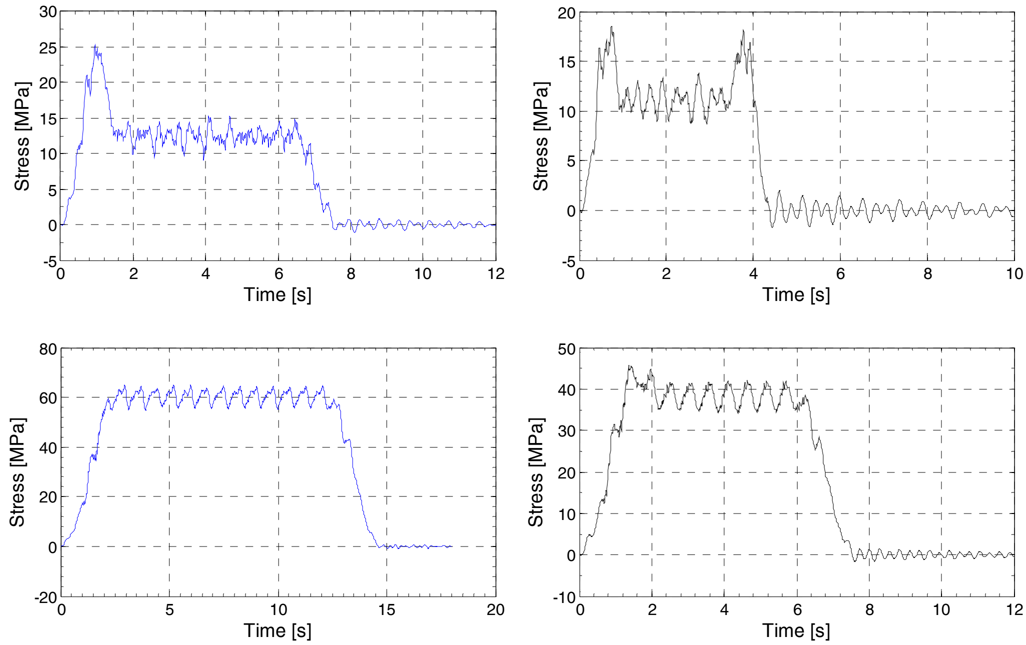


Figure 6. Stress history of the lower chord at midspan for: (a) type 1 train, (b) type 3 train, (c) type 6 train, (d) type 7 train.

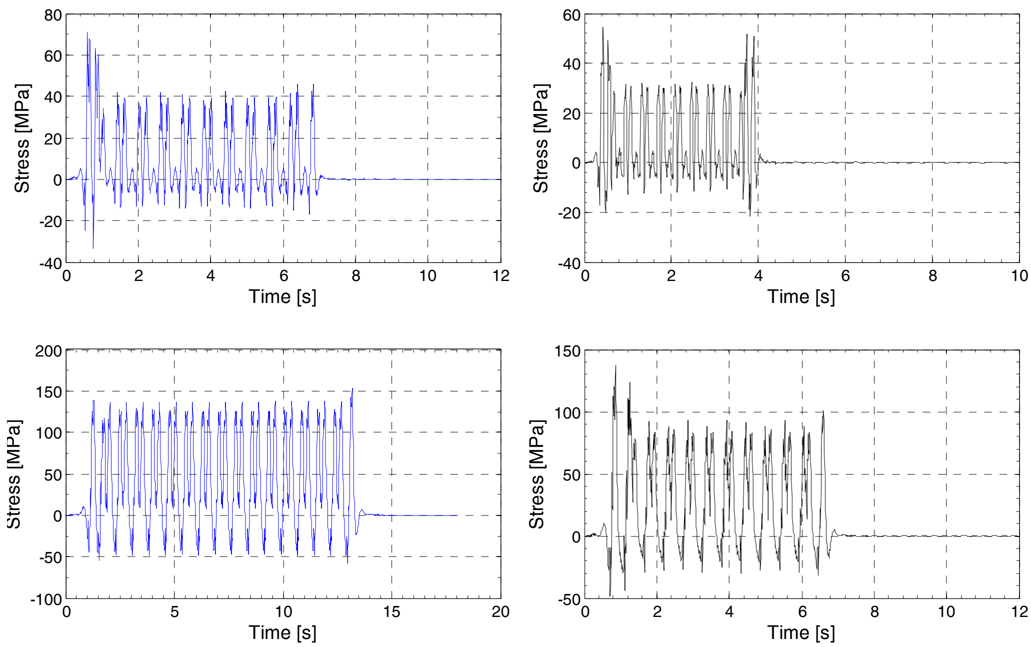


Figure 7. Stress history of the stringers for: (a) type 1 train, (b) type 3 train, (c) type 6 train, (d) type 7 train.

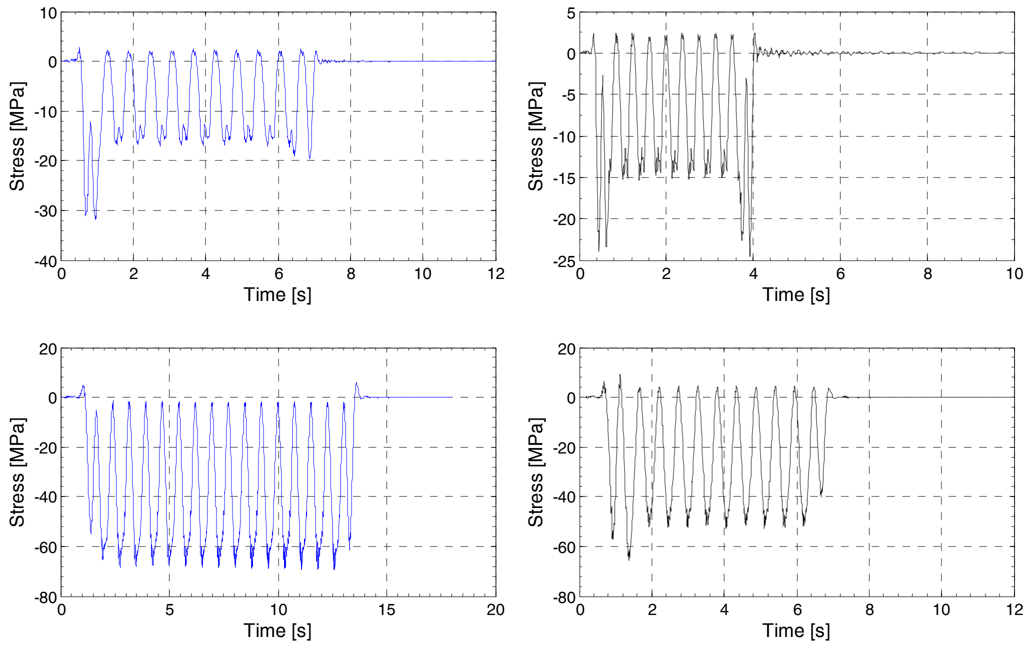


Figure 8. Stress history of the floor-beam for: (a) type 1 train, (b) type 3 train, (c) type 6 train, (d) type 7 train.

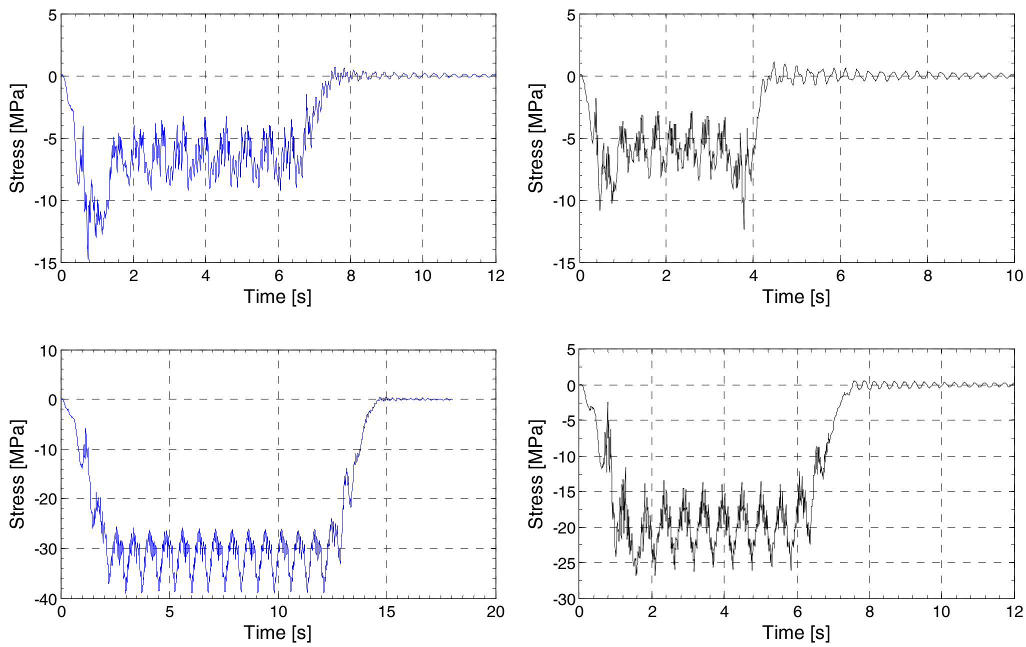


Figure 9. Stress history of the bracing for: (a) type 1 train, (b) type 3 train, (c) type 6 train, (d) type 7 train.

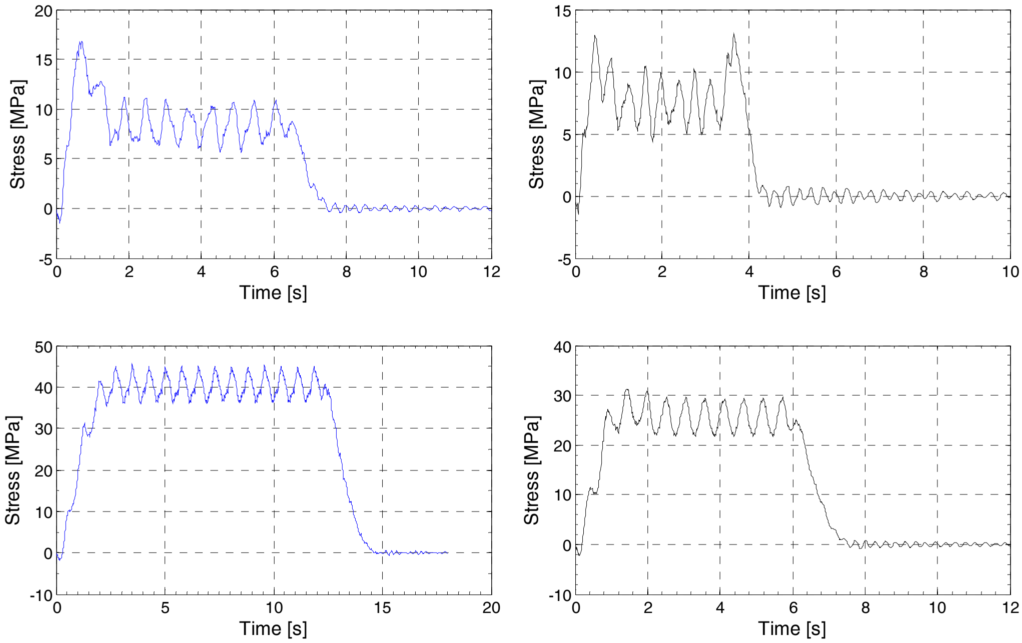


Figure 10. Stress history in a stiffening plate for: (a) type 1 train, (b) type 3 train, (c) type 6 train, (d) type 7 train.

reference values $\Delta\sigma_C$ and $\Delta\tau_C$ for the fatigue strength at 2 million cycles. For riveted connections, many authors [1] and the Italian railways company [24] suggest a value $\Delta\sigma_C$ = 71 N/mm^2 in design. Adopting this value, the fatigue strength is obtained starting from the extended fatigue strength curves defined as follows:

$$(\Delta\sigma_R)^m N_R = (\Delta\sigma_C)^m 2\cdot 10^6 \text{ with } m\text{=3 for } N \le 5\cdot 10^6 \tag{3}$$

$$(\Delta\sigma_R)^m N_R = (\Delta\sigma_C)^m 2\cdot 10^6 \text{ with } m\text{=5 for } 5\cdot 10^6 \le N \le 1\cdot 10^8 \tag{4}$$

$$\Delta\sigma_L = (5/100)^{1/5}\Delta\sigma_D \text{ (cutoff limit)} \tag{5}$$

Information in design standards is sometimes insufficient for a reliable estimation of fatigue behavior (Remaining Fatigue Life - RFL) of the riveted details in old steel bridges. In most cases, standards give the lower boundary of the test results for a set of specified riveted details, which is not always reasonable when estimating RFL. In [1], S-N lines and their parameters are proposed for frequently used riveted details in old steel bridge riveted connections. The influence of mean stresses and stress ratio on fatigue strength category is taken into account by function $f(R)$ as follows:

$$\Delta\sigma_c(R) = f(R)\cdot\Delta\sigma_{c,0} \tag{6}$$

where $\Delta\sigma_c(R)$ is the fatigue strength category for the considered detail as a function of the stress ratio R, $\sigma_{c,0}$ is the fatigue strength category for the considered detail for stress ratio R = 0 and f (R) is the correction function which, for mild steels (low carbon steels with carbon level under 0,25%), is determined by the following formula [1]:

$$f(R) = \begin{cases} \dfrac{1-R}{1-0.6R} & when\ R > 0 \\ \dfrac{1-R}{1-0.4R} & when\ -1 \leq R \leq 0 \end{cases} \tag{6}$$

Using Eurocode S-N line model and stress ratio correction $f(R)$, number of cycles to failure (fatigue crack initiation) may be obtained as a function of stress range and stress ratio R as follows.

$$(\Delta\sigma_R)^m N_R = (f(R)\Delta\sigma_{C,0})^m 2\cdot 10^6 \quad \text{with } m\text{=5 for } N \leq 1\cdot 10^8 \tag{7}$$

$$\Delta\sigma_L = (2/100)^{1/5}\Delta\sigma_D \text{ (cutoff limit) for } N \geq 1\cdot 10^8 \tag{8}$$

A stress history is determined from dynamic analysis calculations of the structural response in each structural element (see section 2.2). Then, according to Eurocode 3 part 1-9, the stress histories are evaluated by the cycle counting methods (or rainflow method) in order to determine the stress ranges and the associated numbers of cycles. The stress range spectrum is determined by collecting the stress ranges and the associated number of cycles in descending order. The damage D_d during the design life (100 years) is then calculated by means of the Miner's linear damage accumulation rule [9, 25]:

$$D_d = \sum \frac{n_E}{N_R} \tag{9}$$

where n_E is the number of cycles associated with the stress range $\gamma_F \Delta\sigma_i$ for band i in the spectrum and N_R is the endurance (in cycles) obtained from S-N curves from a stress range $\gamma_F \Delta\sigma_i$. The fatigue limit is based on damage accumulation by means of the following criterion:

$$D_d < 1{,}0 \tag{10}$$

Finally, the remaining fatigue life (RFL) of the member/detail is calculated as follows [1, 25]:

$$RFL = \frac{1-D_p}{D_{1f}} \tag{11}$$

where D_p is the damage accumulated in the considered element in past periods of time and D_{1f} is the damage accumulated for one year future exploitation.

5.1 Results

Dynamic analyses of detailed models are performed by using ANSYS finite element program, with a MATLAB code implemented to process all the data (in terms of stress histories and fatigue verifications). For the dynamic analysis, a time step of 0.01 seconds is chosen and all the traffic mixes defined in Eurocode 1 are investigated. The design life of the viaduct is assumed 100 years (as recommended in Eurocode 3 – part 1-9). As for the evaluation of the Remaining Fatigue Life, the past life of the old bridge is assumed 64 years.

Two different models for determining the fatigue strength of the investigated details are used:

- Fatigue strength model (F1), based on Eurocode [9] and Eqs. (3-5). The detail category for the investigated details is 71. The influence of stress ratio and mean stresses is neglected;
- Fatigue strength model (F2), based on Eurocode, but with the influence of stress ratio using the correction function $f(R)$ (Eqns 6-8).

Result are given for 7 selected points:

- Points no. “1” and “2” are located in the bottom chord, as indicated in Figure 11a;
- Point no. “3” is on the stringer, at midspan, placed in the bottom chord, as indicated in Figure 11a;
- Points no. “4” and “5” are on the floor-beam, see Figure 11b;
- Points no. “6” and “7” are on the main bracing system, Figure 11a.

The investigated points are selected considering the results obtained in the global fatigue analysis. Damage indexes over 100 years are reported in Figure 12; it is show that for point “1” and “2” the damage indexes are $D_d = 0$. All stiffening plates introduced in the detailed local model are subject to cycles over the cutoff limit.

Damage index for point “3” is very high, with value of about 15. For Point “3”, it is interesting compare the procedure defined in Eurocode (Fatigue strength model F1), with that proposed by Georgiev (Fatigue strength model F2, [1]). Due to many cycles with stress ratio less than zero (and consequently with values of function $f(R)$ moderately greater than 1), the damage index decrease from 15.4 to 8.80. The Remaining Fatigue Life (RFL) is respectively -57 years and -31 years. Finally, as far as the bracing system is concerned (points n. 6 and n. 7), only small-amplitude stress histories due to both local vibrations and global effects are found. The maximum stress is under the cutoff limit; for this reason, the Damage index is zero and the RFL is infinity.

The values of Damage indexes obtained from Fatigue Strength models F1 and F2 in stringers totally disagree with present investigations carried out on the real structure. In fact, after over 64 years of service, no visible fatigue damages have been detected in any element or connection. This is probably due to the smaller fatigue loading spectra acting on the bridge than those defined by the Eurocode.

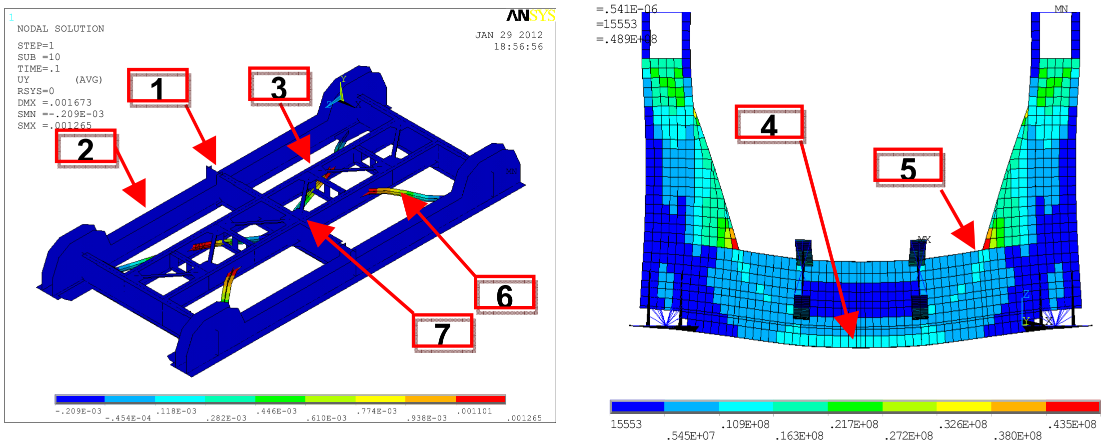


Figure 11. Monitored points for fatigue assessment.

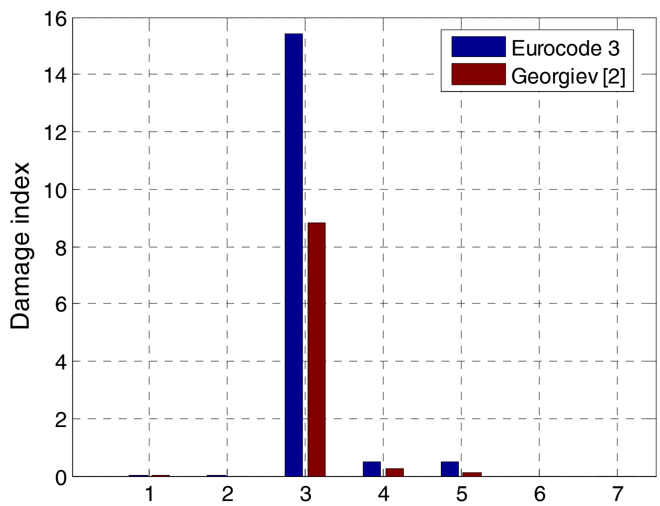


Figure 12. Damage index for Fatigue strength model F1, based on Eurocode 3, and for Fatigue strength model F2 [1]

6 Concluding remarks

A global finite element model of the bridge has been built and calibrated using experimental dynamic data (first four natural frequencies and mode shapes). The global model is used to perform a preliminary fatigue analysis. The fatigue assessment is carried out following the procedure defined in the section 6 of EN1991 – Eurocode 1 [10]. At the same time, the global model is used to find if and where local vibrations may occur in structural members. After having identified the members potentially subject to fatigue or local vibrations, a substructure finite element model containing the specific member has been studied. The detailed substructure model has been then excited by prescribing at the boundaries the displacement time histories obtained in the corresponding nodes of the global model. A linear dynamic transient analysis has then been performed. From the stresses obtained in the detailed finite element model, the fatigue phenomena are studied using both the classical Miner relationship and the procedure described in [4].

The main lower bracing system is identified as the one more prone to local vibrations. Numerical results show only small-amplitude stress histories arising from local vibrations and the maximum stress is lower the cutoff limit. Critical members and details for fatigue are found in stringers, due to high stress range rather than vibration induced and/or distortion induced. All other details seem to be not critical for fatigue.

Values of damage indexes obtained from fatigue strength model F1 and F2 in stringers totally disagree with investigations carried out on the real structure. In fact, after over sixty-four years of service, no visible fatigue damages have been found in any element or connection.

In a future research, a long-term experimental campaign will be performed and vibrations measured on the bracings will be compared to those obtained from numerical evaluations. Stresses on stringers, floor-beams and connections will also be monitored. As far as the damage index of the most critical elements is concerned, during the long-term monitoring masses of trains will be also measured in order to obtain better estimates of the damage index adopting real loading spectra.

Acknowledgements

The research was developed in the framework of the FADLESS research project (Fatigue Damage Control and Assessment for Railway Bridges). Financial supports of European Commission - RFCS (Research Fund for Coal and Steel) is gratefully acknowledged.

The second author is presently a fellow of the Italian Academy of the Columbia University in New York: the support of the Italian Academy if gratefully acknowledged.

References

[1] L. Georgiev. “Fatigue life analysis of an old riveted bridge based on S-N lines”. Proceedings of “7th International Conference on Bridges across the Danube”. Sofia, Bulgaria October 14-15, 2010.

[2] B.M. Imam, T.D. Righiniotisa, M.K. Chryssanthopoulos, “Numerical Modelling of Riveted Railway Bridge Connections for Fatigue Evaluation”, Engineering Structures, 29(11), 3071-3081, 2007.

[3] R.I. Stephens, A. Fatemi, R.R. Stephens, H.O. Fuchs, “Metal Fatigue in Engineering”, 2nd edition, John Wiley & Sons, Inc., New York, 2001.

[4] N.M.M. Maya, J.M.M. Silva, “Theoretical and experimental modal analysis”, Research Studies Press Ltd, Taunton, England, 1997.

[5] D.J. Ewins, “Modal Testing: Theory and Practice”, New York, John M. Wiley & Sons, 2000.

[6] M. Savoia, L. Vincenzi, “Differential evolution algorithm for dynamic structural identification”. Journal of Earthquake Engineering, 12(5), 800-821, 2008
[7] L. Vincenzi, M. Savoia, “Coupling of Differential Evolution algorithm and Quadratic Approximation for dynamic identification”, Proceedings of “ICOSSAR - International conference of structural safety and reliability”, Osaka, Japan, 2009.
[8] L. Vincenzi, M. Savoia, “Improving the speed performance of an evolutionary algorithm by second-order cost function approximation”. Proceedings of “2nd International Conference on Engineering Optimization”, September 6-9, Lisbon, Portugal, 2010.
[9] EN 1993-1-9, “Eurocode 3: Design of steel structures –Part 1-9: Fatigue”, 2004.
[10] EN 1991-2, “Eurocode 1 - Actions on structures - Part 2: Traffic loads on bridges”, 2003.
[11] C. Rainieri, G. Fabbrocino, G.M. Verderame, E. Cosenza, “Modal analysis of the Tower of the Nations in Naples”, Proceedings of “XIV World Conference on Earthquake Engineering”, Beinjin, China, 2008.
[12] L. Zhang, T. Wang, Y Tamura, “A Frequency-Spatial Decomposition (FSDD) Technique for Operational Modal analysis”. In Proceedings of the “1st International Operational Modal Analysis Conference (IOMAC)”, April 26-27, Copenhagen, Denmark, 2005
[13] B. Cauberghe, “Applied frequency-domain system identification in the field of experimental and operational modal analysis”. PhD thesis, Vrije Universiteit Brussel, 2004
[14] R. Pintelton, J. Schoukens, “System Identification – a Frequency Domain Approach”, IEEE Press, New York, 2001.
[15] ANSYS, “Robust simulation and analysis software”, Available at: http://www.ansys.com Release 10.0 ANSYS Incorporated, 2001.
[16] G.E.P. Box, K.B. Winson, “On the experimental attainment of optimum conditions”. J. R. Statist. Soc., B13, 1-45, 1951.
[17] A. Khuri, J.A. Cornell, “Response Surfaces. Designs and Analyses”, Marcel Dekker Inc., New York, 1996.
[18] R. Storn, K. Price, “Differential Evolution - a simple and efficient heuristic for global optimization over continuous spaces”. J. of Global Optimization, 11(4), 341-359, 1997.
[19] C. Gentile, F. Matinez y Cabrera, “Dynamic assessment of a curved cable-stayed bridge at the Malpensa 2000 Airport, Milan, Italy”. Structural Engineering International, 1, 52-58, 2001.
[20] L. Fryba, “Dynamics of railway bridges” Co-published with Thomas Telford Ltd, London, 1996.
[21] K. Liu, G. De Roeck, G. Lombaert, “The effect of dynamic train-bridge interaction on the bridge response during a train passage”. Journal of Sound and Vibration 325, Elsevier, 2009.
[22] L. Vincenzi, M. Savoia, F. Rossi, “Vehicle-bridge interactive analyses on the Lagoscuro viaduct”, Proceedings of the “CST2012 conference”, Dubrovnik,

Croatia, 2012.
[23] L. Vincenzi, M. Savoia, W. Salvatore, “Experimental modal analysis and fatigue assessment on the Lagoscuro viaduct”, in Proceeding of “IABMAS 2012”, Como Lake, Italy, 2012
[24] RFI – Rete Ferroviaria Italiana, “Istruzioni n° 44F per le verifiche a fatica dei ponti ferroviari” (in Italian), 2009.