

Weigh in Motion Measurement and Experimental Fatigue Assessment of a Railway Bridge

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Abstract

Fatigue represents one of the most diffused failure modes in steel railway bridges. Phenomena such as “vibration induced” and “distortion induced” fatigue are still only partially covered by design codes and represent critical aspects for the assessment of existing bridge and the design of new ones. The European research project FADLESS “Fatigue damage control and assessment for railways bridges” is directed towards defining innovative technical guidelines for the assessment and control of existing and new bridges with regard to fatigue phenomena induced by vibrations and distortions.

In this paper, the experimental methodology applied for the fatigue assessment of an Italian case study, the Panaro Bridge, is reported. Such a methodology consists of: preliminary execution of the fatigue assessment of the bridge; experimental structural characterisation; installation of a developed structural monitoring system including a properly designed weigh in motion system.

Keywords: structural monitoring system, weigh in motion, distortion fatigue, vibration fatigue, railway bridge, equipped sleeper.

1 Introduction

Fatigue and fracture phenomena represent very important failure modes in railway bridges since such structures experience a very high number of stress cycles during their service life.

Indeed, in spite of many study performed in the past in order to evaluate the fatigue resistance of steel and steel-concrete composite bridges that represented the base for modern codes and standards [1] [2] [3] [4], the increase of railway traffic both for passenger and freight trains, the introduction of new structural materials, the development of new train typologies, the presence of effects like distortion in bridge members and local vibrations, concur to render, even today, fatigue, one of the main

issues to be solved in the design of the new steel railway bridges and the assessment of the existing ones.

From the structural point of view, above mentioned problems together with the possible presence of new train types (characterized by different values of masses, different suspension systems and different operating velocities) lead to introduce new uncertainties to actual design load models for the safety assessment of a steel and steel-concrete composite railway bridges, highlighting the need of a critical review of actual standard codes in order to understand their effectiveness with regards to the new operating conditions.

As a matter of the fact, even if traditional design approaches assumed that a proper detailing and the removing of secondary stresses could limit the application of enhanced analysis methodologies, fatigue damages still affect steel bridges.

In particular, fatigue cracks are caused by distortion phenomena localized in the cross section elements and in the webs due to an out-of-plane deformation that induced localized bending stresses [6]. Referring to existing bridges, transverse stiffeners and connecting plate elements are not welded to the upper flange of welded I-girders and box girders in order to avoid fatigue cracking [7].

In addition to distortion phenomena, also vibration effects, typically due to low local stiffness distribution of some details, affect existing bridges exciting local vibration modes, within the frequency range of 0-30 Hz, during the train passages, so resulting in high local “modal” stresses and a reduction of remaining fatigue life.

Main objective of the European research project FADLESS "*Fatigue damage control and assessment for railways bridges*", funded by the European Research Found for Coal and Steel (RFCS), was to analyze the aforementioned uncertainties on the structural behaviour and integrity of steel and composite steel-concrete bridges with particular attention to the fatigue phenomena. To this aim, enhanced numerical and experimental methodologies for the evaluation of the present fatigue load spectra acting on the railway lines were developed and applied to selected case studies, representative of the bridge types most exposed to such damages, evaluating strength of main elements and critical details by suitable experimental tests.

In the research project, the actual fatigue loading spectra were experimentally measured by the development of suitable long term monitoring systems and also evaluated by the estimation of the expected traffic flows, taking into account of the global dynamic train-bridge interaction effects and the local dynamic detail behaviour and distortional influence on strain patterns. As results of such studies, it was so possible to setup suitable strategies for the evaluation of vibration induced and distortion induced fatigue damaging in steel and composite steel-concrete railway bridges, as well as for the control of the crack growth.

In the present paper, experimental methodology applied for the fatigue assessment of the Panaro bridge, one of the selected case of study, is reported. Such methodology consists in: i) a preliminary execution of the fatigue assessment of the case of study in order to evaluate critical details to be monitored; ii) the execution of global/local structural characterization; iii) the installation of a suitable developed structural monitoring system including properly designed weight in motion apparatus. Objectives of these analyses are the evaluation of real fatigue loading spectra, the measure of the train/bridge interaction forces by the instrumented

sleepers, and the evaluation of the global/local structural response through the processing of the data extracted from the monitoring system for the estimation of the remaining fatigue life of the bridge. At last, results constitute the base for the final developments of the research, leading to a better comprehension of vibration and distortion fatigue phenomena induced by real traffic on steel and steel-concrete composite railway bridges.

2 The Panaro Bridge

The Panaro Bridge (Figure 1) is a bow string steel bridge along the Bologna - Verona railway line.



Figure 1: the Panaro Bridge.

The bridge span length is equal to 75.6 meters and the height of cross section of the bridge varies from 7 to 11 meters. Its structure, composed by welded elements, consists in lower and upper strings, vertical and diagonal beams, main cross girders, lower and upper bracing diagonals and local track bracing system. All joints between the structural members are realized adopting riveted solutions. Support system is composed by two fixed steel bearings and two unidirectional steel bearings, as reported in the Figure 2.

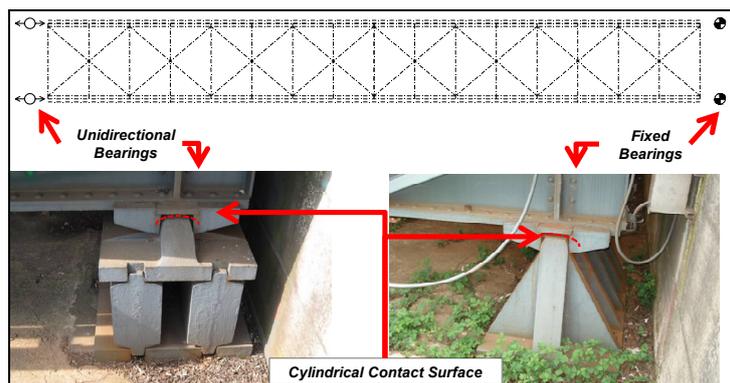


Figure 2: supporting scheme of the Panaro Bridge.

During its service life, the Panaro Bridge was subjected to an heavy railway traffic and some of its secondary elements showed fatigue damages due to distortion and vibration phenomena (Figures 3a and 3b).

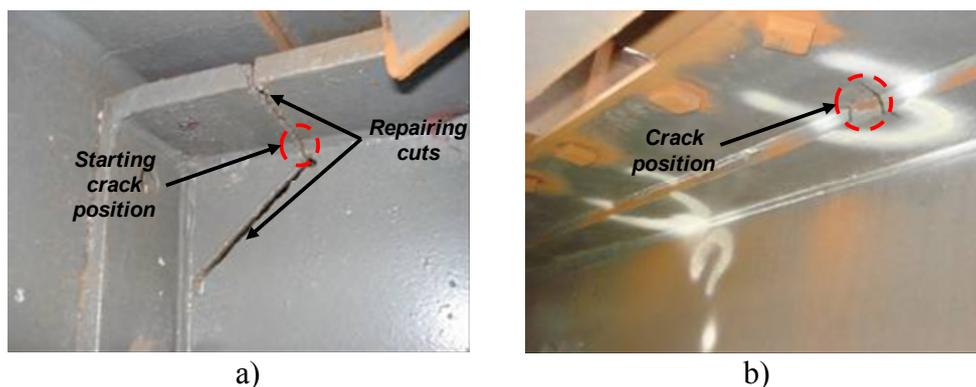


Figure 3: fatigue damages due to distortion and vibration phenomena, Panaro bridge.

In particular, as reported in the Figure 3a, all the track bracing cross beams experienced the same fatigue crack located near its connection to the secondary longitudinal girders. In order to stop the crack propagation, Italian Authority decided to cut the profiles and to remove external bolts, changing the scheme of the connection joint between the damaged components and the longitudinal girders from fully fixed to hinged. In the Figure 3b, the typical fatigue damage found in the welding between the secondary longitudinal girders and the connection plates supporting the wooden sleepers is showed.

3 Preliminary analyses

In order to deeply analyze the structural behaviour of the Panaro Bridge for the FADLESS project purposes, both experimental and numerical analyses were preliminarily performed.

3.1 Fatigue assessment of the Panaro bridge

For the fatigue assessment of the Panaro Bridge and the identification of the critical details to be monitored, global and local numerical models were developed [8].

Two finite element beam models were developed by the ANSYS[®] software: the first model represents the original resistant scheme of the bridge, characterised by the track bracing cross beam fully fixed to the longitudinal girders (Fully fixed model); the second represents the actual resistant scheme of the bridge, where connections at the end of the restraint of the secondary cross beam are hinges (Hinge model), as showed in the Figures 4a and 4b.

In accordance with the Eurocode 1 prescriptions [9], stresses in the relevant structural elements were evaluated by the execution of an incremental dynamic analysis with moving loads using Standard and Heavy traffic mix corresponding to a life cycle of the bridge equal to 100 years and fatigue damages were calculated adopting Palmgren-Miner linear damage accumulation law. In particular, fatigue classes of welded members were assumed according to the indications of Eurocode

[1], while for riveted components the fatigue classes proposed by Taras and Greiner were considered [10].

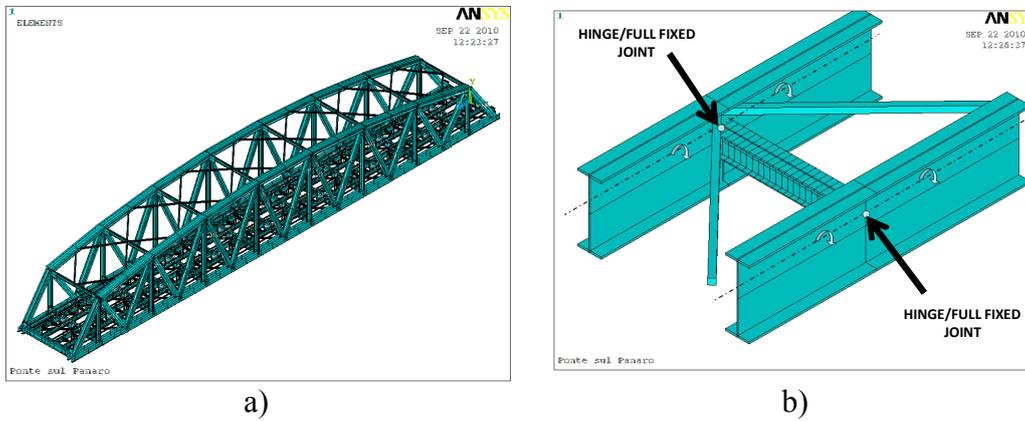


Figure 4: a) finite element global model and b) restraint modification of the secondary cross beam

Main results of the performed analyses, indicated in the Figures 5 and 6, showed that, both for the fully fixed model and for the hinge model, damage indexes were greater than 1, i.e. elements not safe in terms of fatigue strength, at the connection joints between vertical beams and lower/upper stringers (actually undamaged), as reported in the Figure 7a.

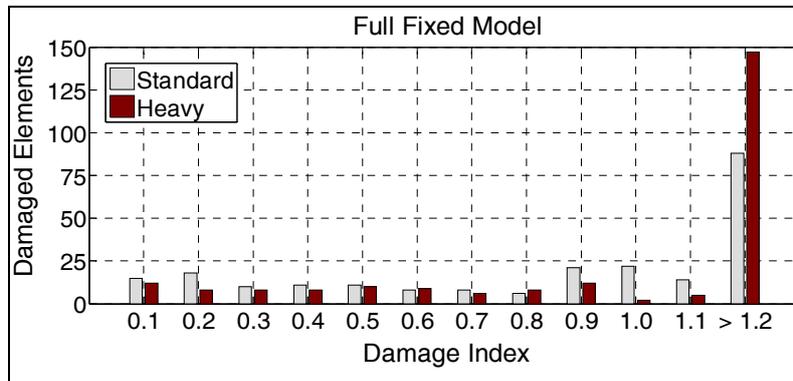


Figure 5: results of the standard fatigue assessment of the fully fixed model – standard and heavy traffic mix

On the contrary, the elements corresponding to the end sections of the track bracing cross beam, resulted damaged only in the fully fixed model, as really occurred, see Figure 7b, but the damage index, nearly equal to 1.2, i.e. a life cycle of 80 years, was not in agreement with the real observed damage. Indeed all of these components presented fatigue cracks just after 30 years.

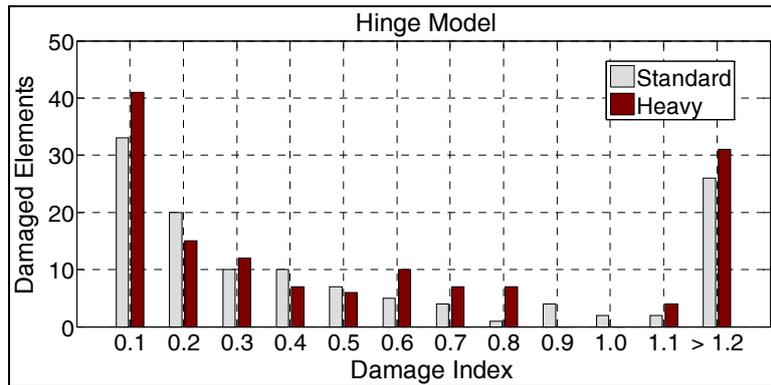


Figure 6: standard fatigue assessment of the hinge model – standard and heavy traffic mix

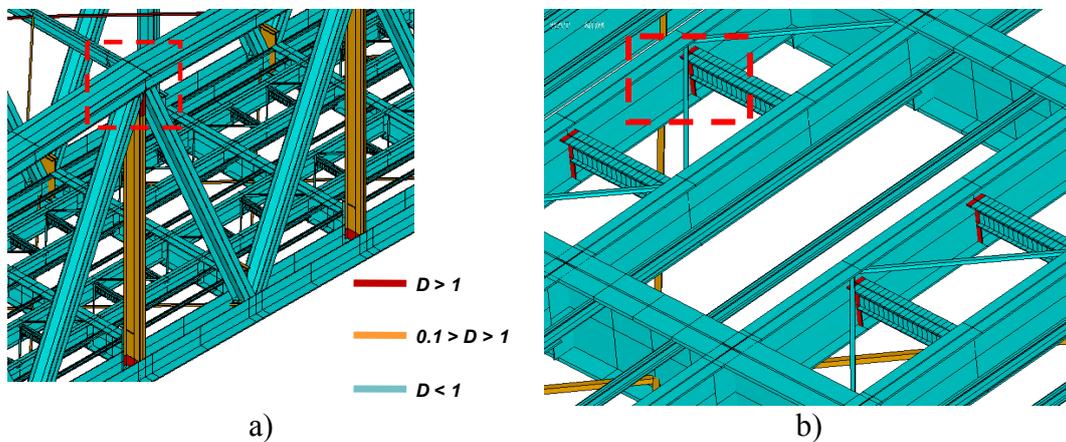


Figure 7: a) joint between vertical beams and lower/upper stringers (hinge model) and b) damaged section of the track bracing cross beam (fully fixed model)

Moreover, using the global bridge model, it was not possible to assess the presence of fatigue cracks observed in longitudinal plate-flange welding of the Panaro bridge due to local features.

As results of fatigue analyses and of real damage observations, the following critical details were selected: i) the track bracing cross beams, ii) the connection plates between longitudinal girders and wooden sleepers and iii) the joints between vertical beams and lower stringers.

3.2 Experimental modal analysis of the Panaro bridge

The experimental analyses of the Panaro bridge were performed in order to develop a suitable characterization of the structure before the installation of the monitoring system.

Acceleration in both vertical and horizontal directions were recorded in correspondence of eight sections of the bridge. Mode shapes were then identified

using Operational Modal Analysis (OMA), applying a multi-step approach and POLIMAX algorithm [11].

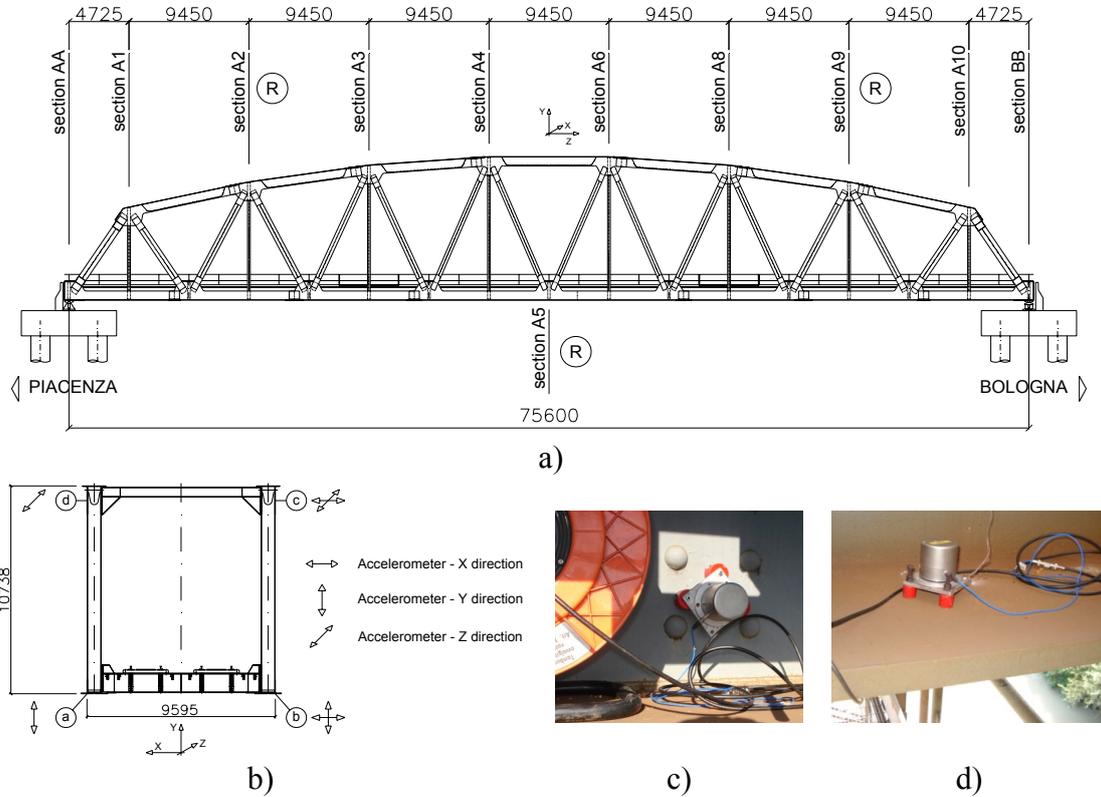


Figure 8: a) and b) layout of the global modal analysis test; horizontal (c) and vertical (d) accelerometers on main lower longitudinal beam.

Accelerations were recorded by PCB[®] accelerometers and LMS[®] Scadas acquisition unit adopting a sampling frequency of 400 Hz for time periods of about 15 minutes. Seven global modes were identified as reported in the table 1. In the Figure 9 shapes of the first two modes are reported.

Mode n°	Frequency[Hz]	Mode Shape
1	1.77	Lateral Bending
2	3.65	Torsional
3	3.68	Vertical Bending
4	4.08	Lateral Bending
5	6.86	Distortional
6	10.93	Torsional
7	12.35	Vertical Bending

Table 1: identified global vibration mode of Panaro bridge.

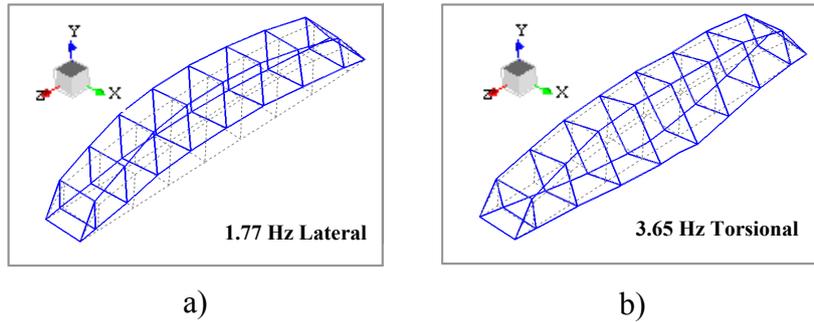


Figure 9: a) the 1st and b) the 2nd identified global mode shapes.

OMA was also applied for the evaluation of local modes of the track bracing cross beams. In this case, by a single acquisition performed with the aforesaid record setting, it was possible to identify three structural modes, as showed in the table 2. In the Figure 10 the first and the second mode shapes are showed.

Mode n°	Frequency[Hz]	Mode Shape
1	15.65	Vertical Bending
2	44.18	Lateral Bending
3	97.22	Vertical Bending

Table 2: identified local vibration modes of the track bracing cross beam.

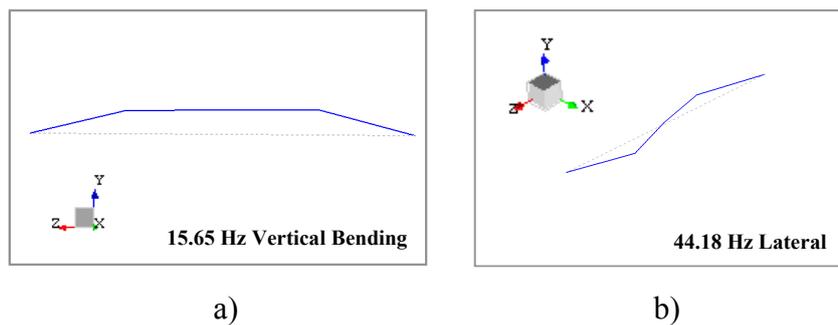


Figure 10: a) the 1st and b) the 2nd mode shapes identified for the track bracing cross beam.

OMA wasn't applicable to the connection plate between longitudinal girders and wooden sleepers, as global modes showed a great influence in such analysis. For this reason, the impact testing technique was applied, as reported in the Figure 11.

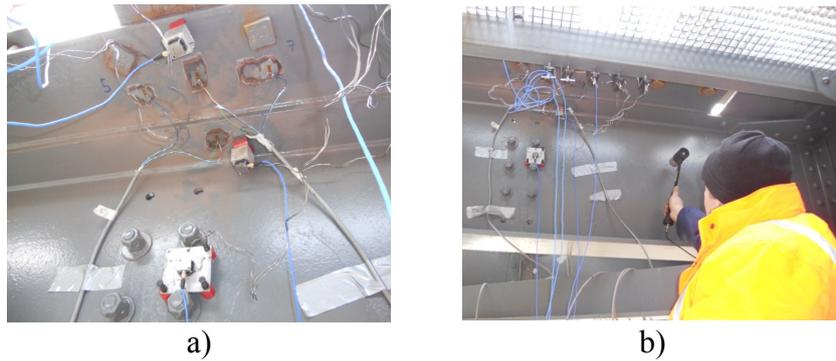


Figure 11: a) layout and b) execution of impact testing on the connection plate between longitudinal girders and wooden sleepers.

Data processing of impact test acquired signals are still ongoing. In the Figure 12 an example of the Frequency Response Functions (FRF's) is reported.

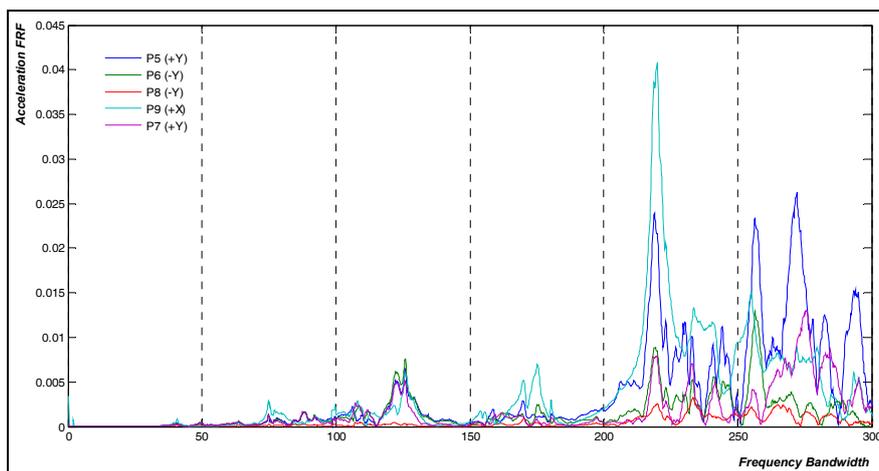


Figure 12: FRF evaluated through the impact test.

4 The weight in motion system

In order to measure the train axle loads acting on the bridge, i.e. the train weighing in motion, an instrumented wooden sleeper was properly designed. This particular system aims at directly measuring the axle load, avoiding measurement of deformation of secondary bridge elements, as for example in weighing in motion system composed by strain gauges located on different points of the bridge or in the system illustrated in [12], where axle loads are evaluated through a resistance strain transducer embedded in the bridge deck.

The first revision of the instrumented sleeper (Rev.1), reported in Figure 13a, was designed and realized for the validation of the proposed measure.

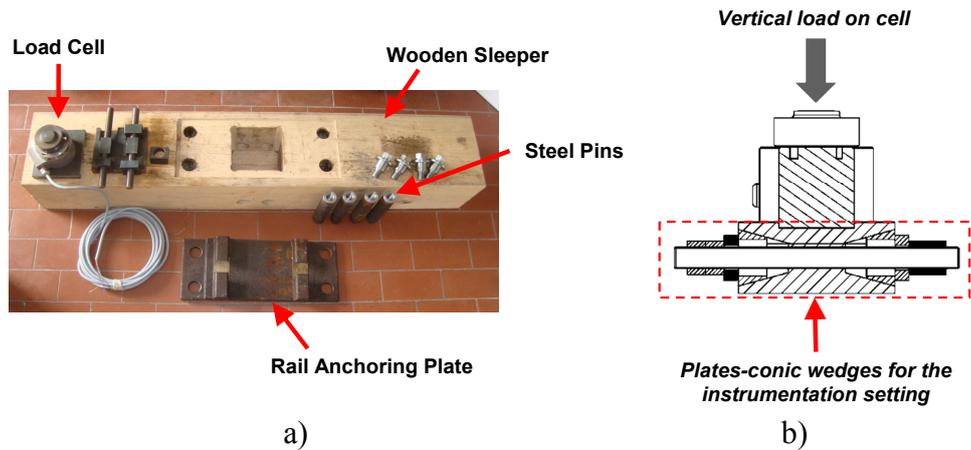


Figure 13: a) Rev.1 of the developed weighting in motion system and b) instrumentation setting system of the load cell

The working principle of such system consists in the possibility of the rail anchoring plate to slide over the four steel pins located in the wooden sleeper, in order to apply the vertical load due to the train wheels directly on a load cell mounted in the wooden sleeper. The load cell is positioned over a setting system composed by four conic wedges and two machined plates, as reported in the Figure 13b. Such system is necessary to guarantee the suitable contact between the load cell and the rail anchoring plate, a basic requirement for this measurement, also taking into account of rail misalignment imperfections.

A validation test of the developed prototype of the Rev.1 was then performed with the following test configuration. The equipped sleeper was mounted between two ordinary sleepers in order to reproduce the real operating conditions. A part of rail was connected to the three sleepers and the load was then applied by the use of an hydraulic jack, as showed in the Figure 14. Measurements of rail strains using electric gauges and measurements of vertical displacement of the load application point on rail using LVDT sensors were performed.



Figure 14: a) global layout of the validation test and b) a particular of the hydraulic jack.

On the basis of the measures of applied load by using the proposed instrumentation, it was possible to observe that such weighing in motion system guarantees a linear response, but a suitable calibration on field is necessary in order to extract correct values of train axle loads, as showed in the Figure 15, which reports the load cycle of one of experimental tests.

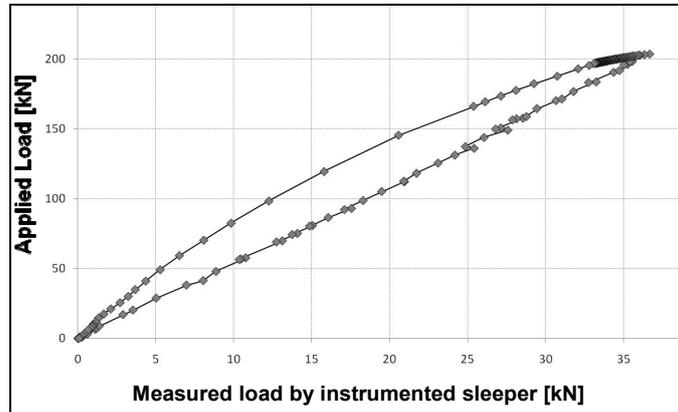


Figure 15: load cycle measured by the validation test

It is important to remark that the difference between Eurocode prescriptions and test results on longitudinal distribution of wheel loads due to the rail, are mainly due to the limitation of test layout.

In order to verify the instrumentation durability, numerical solid models of all the components were developed through ANSYS[®] software. Static analyses of these steel components performed through the application of loads foreseen in Eurocode 1 [8], highlighted the needs of a design review to reduce stresses and to avoid fatigue damage. Maximum Von Mises equivalent stress in the upper steel plate of the system was equal to 270 MPa, see Figure 16, greater than the fatigue limit for steel grade S375 equal to 220 MPa.

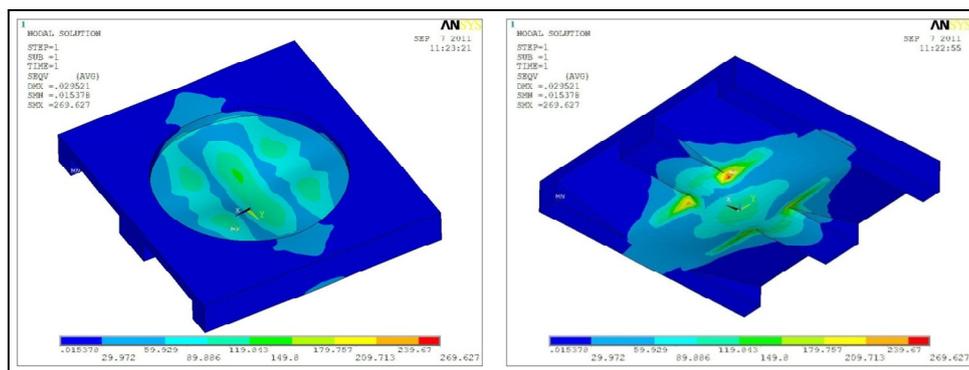


Figure 16: numerical model of the upper steel plate of the Rev.1 setting system.

To these purposes, a new revision (Rev.2) of the instrumentation was developed. Main changes focused on the instrumentation setting system composed by plates and

conic wedges, as showed in the Figure 17. Static analyses performed on the Rev.2 showed a general decrease of stresses. Maximum Von Mises equivalent stress in the upper plate of the setting system was equal to 105 MPa, lower than fatigue limit, see Figure 18.

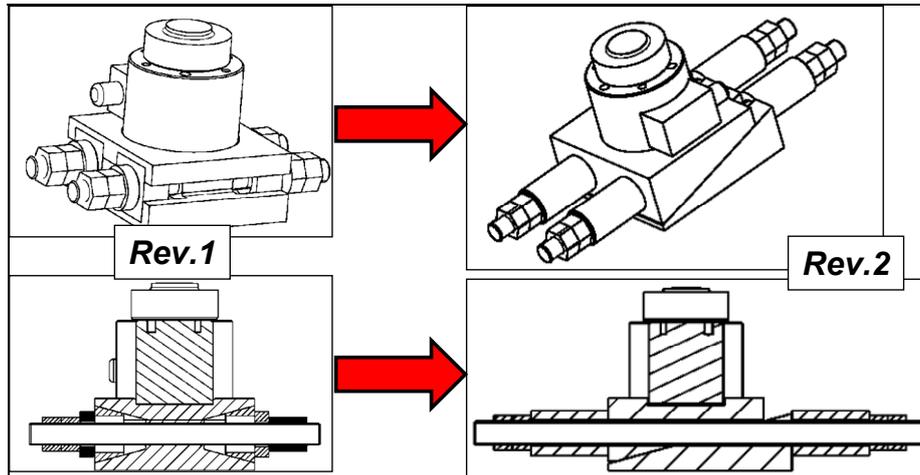


Figure 17: main changes in instrumentation setting system.

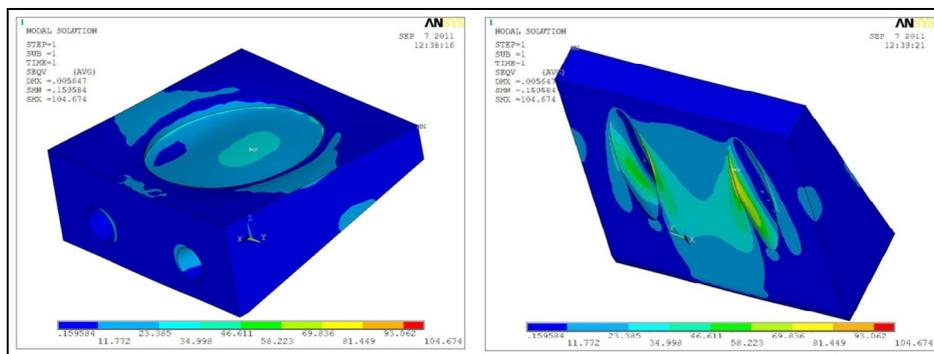


Figure 18: numerical model of the upper steel plate of the Rev.2 setting system

Other changes to the Rev.1 were applied in the development of the Rev.2 in order to guarantee the structural integrity of the wooden sleeper. In particular two lateral and one lower steel stiffening plates were added in order to avoid excessive wood deformations in operative conditions.

Finally, final version of the developed equipped sleeper was tested adopting the same test configuration of Rev.1, as showed in the Figure 20.

Rev.2 of the instrumented sleeper was approved by Rete Ferroviaria Italiana S.p.A., the Italian railway infrastructures manager society, for the installation on the Panaro bridge, without any restriction on train velocities.

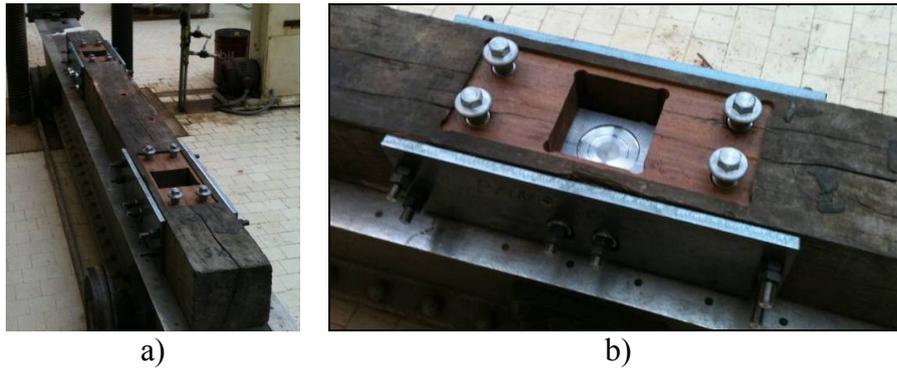


Figure 20: Rev.2 of the instrumented sleeper

5 The long-term monitoring system

A long-term monitoring system was developed in order to individuate the actual fatigue loading spectra, to evaluate the train axle loads acting on the bridge, train composition and train velocities, and to measure the interaction forces between train and bridge. Such system was also designed to allow the analysis of local response of identified critical details under the train passages, in order to analyze the effects of distortions and local vibrations on the fatigue resistance.

The layout of the long-term monitoring system is reported in the Figure 21.

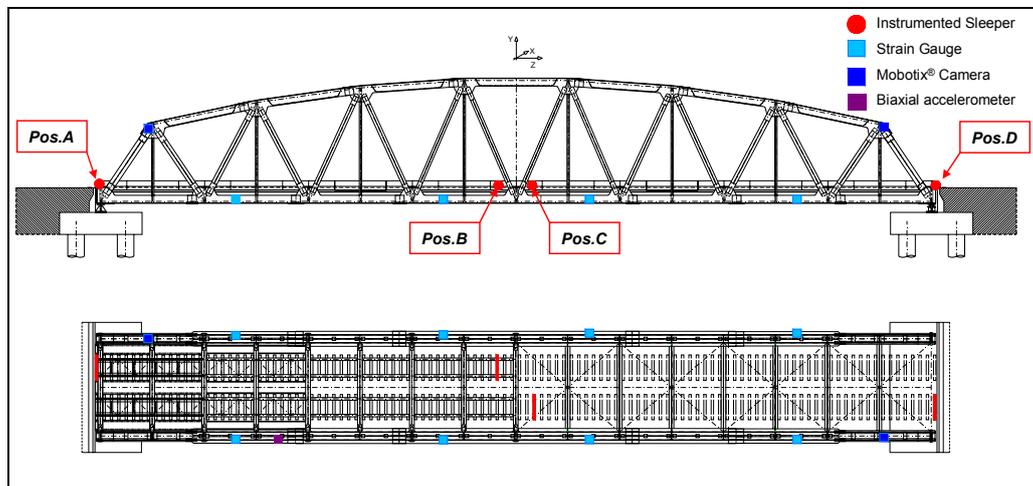


Figure 21: layout of the developed long-term monitoring system.

Such system is composed by: four instrumented sleepers, for the evaluation of train axle loads, train compositions and train velocities (Position A and Position D) and for the evaluation of train/bridge dynamic interaction forces (Position B and Position C); strain gauges installed on the main lower longitudinal girders for the evaluation of global effects on the bridge due to train passages; a biaxial accelerometer located on the quarter span length of the main lower longitudinal girder, for the evaluation

of global mode frequencies during the monitoring period analysing free vibrations after a train passage; two Mobotix[®] cameras for the visual estimation of train types; a National Instruments[®] acquisition unit connected to the aforesaid sensors types and controlled by an host computer via local network. In the Figure 22 a graphical representation of the local network installed on the bridge is reported.

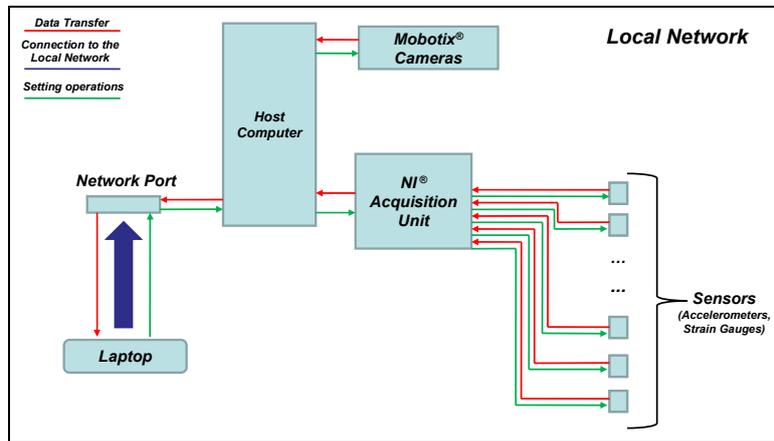


Figure 22: monitoring system local network

Data acquisition and data storage are executed by a LabVIEW[®] procedure performed by the acquisition unit. This procedure is composed by the running of three parallel tasks: with reference to the Figure 23, after the buffer initialization (Point 0), the first task (Point 1) consists in the acquisition of sensor data with a sampling frequency of 800 Hz and in their storage in the buffer; the second task (Point 2) consists in the buffer reading every half second, writing in a file every 800 samples; and the third task (Point 3) is composed by the file transfer to the host computer deleting the original file on the acquisition unit after file transfer itself.

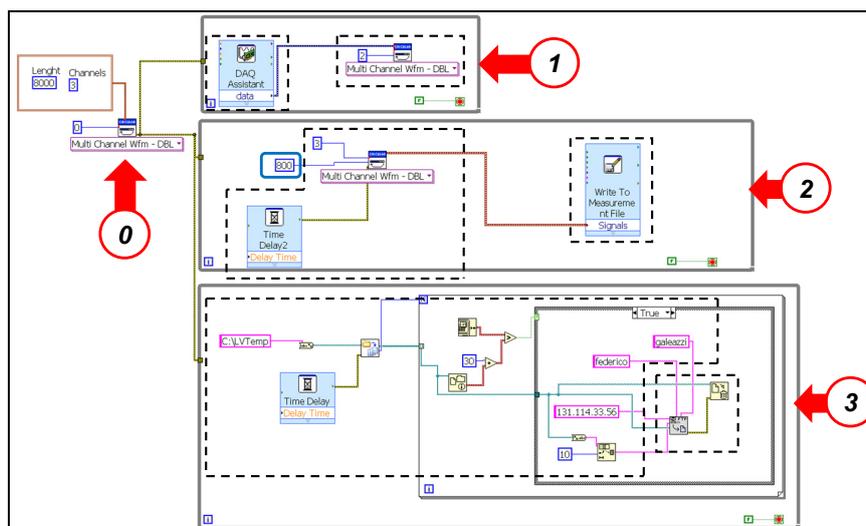


Figure 23: LabVIEW[®] procedure performed by the acquisition unit

In order to evaluate local dynamic behaviour and distortional strain patterns under train passages in critical details (i.e. vibration/distortion induced fatigue), the monitoring system foresaw the installation of three accelerometer and eight strain gauges for the local monitoring of the track bracing cross beam and the connection plate between longitudinal girders and wooden sleepers, as reported in the Figure 24, and the installation of six strain gauges for local monitoring of the joint between vertical beams and upper stringers, as reported in the Figure 25.

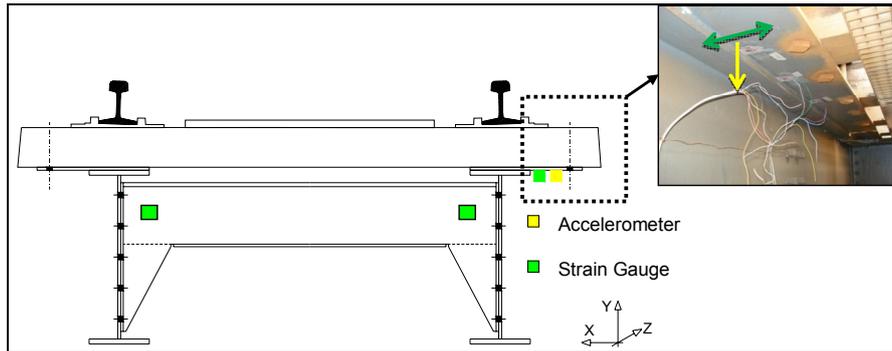


Figure 24: local monitoring system of the track bracing cross beam/connection plate between longitudinal girders and wooden sleepers.

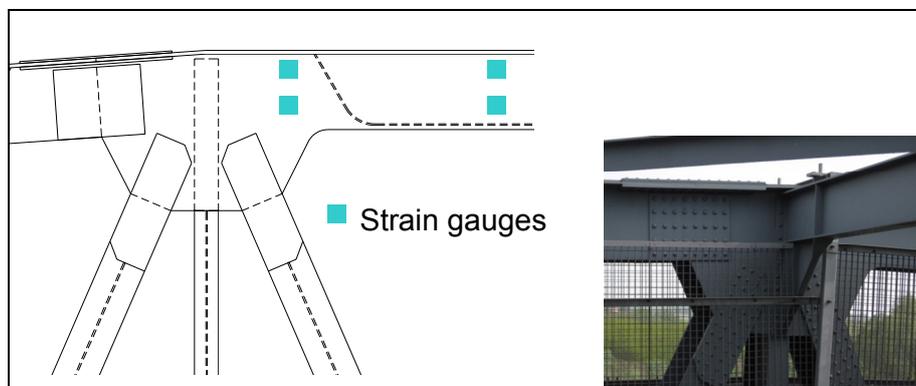


Figure 25: local monitoring system of the joint between vertical beams and upper stringers.

6 Preliminary results and outlook

The complete installation of the developed long-term monitoring system is still ongoing, only the strain gauges were mounted. In the Figure 26 such installations are represented.

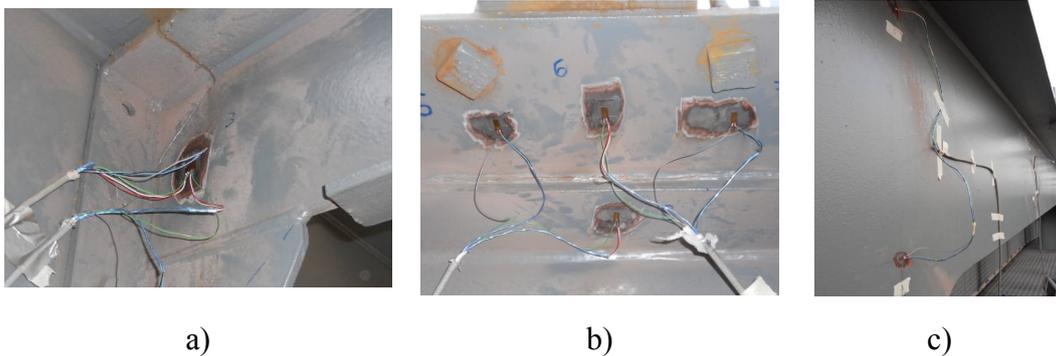


Figure 26: preliminary installation of local monitoring system for a) the track bracing cross beam, b) the connection plate between longitudinal girders and wooden sleepers, and c) joint between vertical beams and upper stringers.

In any case, some interesting results were already extracted from mounted sensors. Indeed, by the analysis of the data extracted from the strain gauges located in the track bracing cross beam, as reported in the Figure 27, it was possible to observe that the local repair intervention on such detail reduced the stress field (the maximum stress variation was equal to about 5 MPa under a freight train passage).

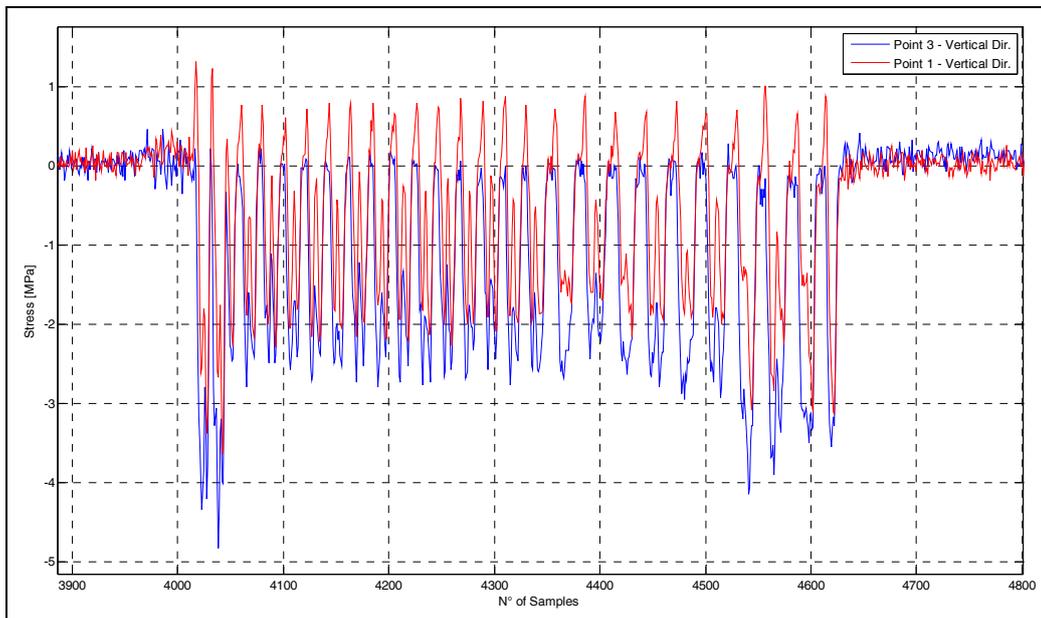


Figure 27: data extracted from local monitoring system installed on the track bracing cross beam – freight train passage

By the analysis of the data extracted from the strain gauges of the local monitoring system developed for the connection plate, it was possible to observe that the welding between the connection plate and the longitudinal girders was subjected to intense stress variations, and then to fatigue phenomena. For example, the maximum

stress variation due to a freight train passage is nearly equal to 45 MPa, as reported in the Figure 28.

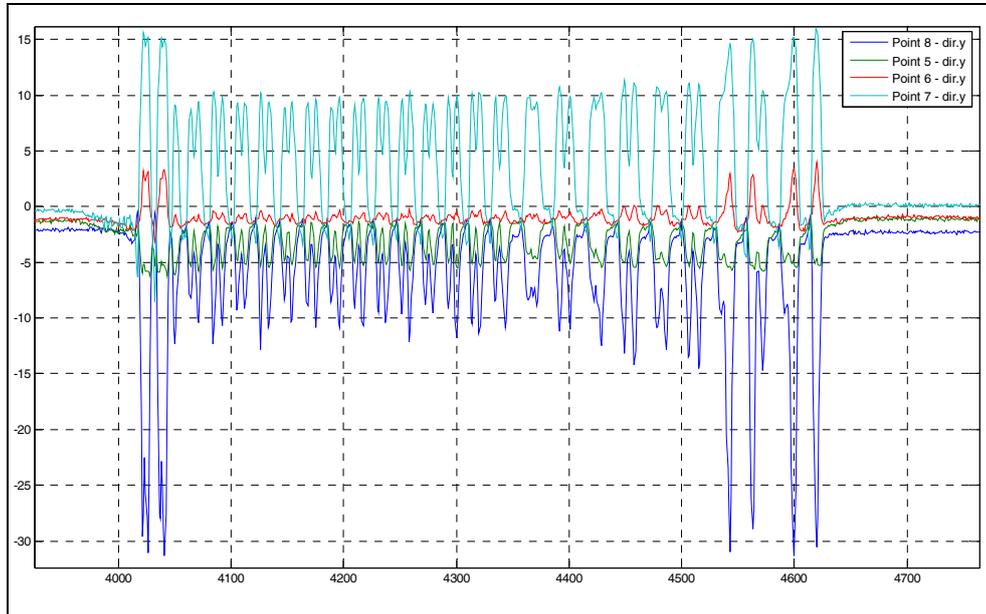


Figure 28: data extracted from local monitoring system installed on the connection plate between longitudinal girders and wooden sleepers – freight train

By the evaluation of data extracted from the strain gauges located on the connection joint between vertical beams and upper stringers, it was possible to analyze the stress field in this details. For example, a peak stress value due to a freight train passage nearly equal to 18 MPa was found, as reported in the Figure 29.

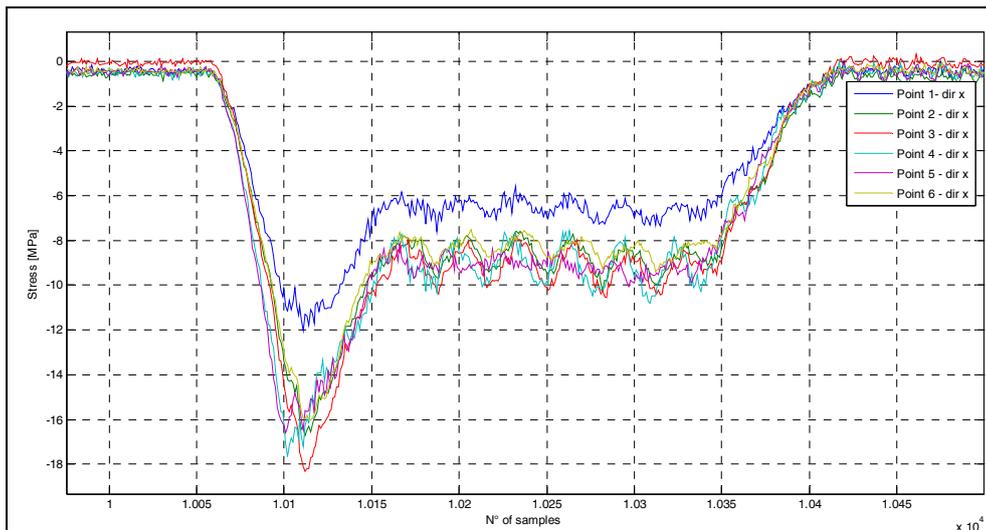


Figure 29: data extracted from local monitoring system installed on the connection joint between vertical beams and upper stringers.

After the complete installation of the monitoring system, as reported upon, it will be possible to pursue main purposes of such system, i.e. the evaluation of real fatigue loading spectra, the measure of the train/bridge interaction forces by the instrumented sleepers, the evaluation of the global/local structural response, and, finally, the estimate of the remaining fatigue life of the bridge.

At last, results constituted the base for the final developments of the research, leading to a critical review of the actual design fatigue loads, together with a better comprehension of vibration and distortion fatigue phenomena.

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