SHM of the new bridge over the River Sado in Portugal during construction

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Summary

The new railway crossing over the River Sado is a 2,7 km long bridge, including two approach viaducts and a main bridge. The main bridge, with a continuous deck over 480 m, consists of three continuous bowstrings with a single plane of hangers on the bridge axis. The bridge is located in an environmentally sensitive area which conditioned the design and the construction methods used.

The bridge importance, its structural complexity and the innovative constructions methods used were the main constraints and motivations for the development of the implemented structural monitoring system, which is described herein. The motivation of this paper is to present the structural monitoring system and the experimental bridge behaviour during the deck incremental launching and the hoisting of the arches.

Keywords: structural health monitoring, bowstring arch bridge, bridge construction, incremental launching

1. Introduction

Structural Health Monitoring (SHM) is an important tool to increase the service life of a structure and improving their safety. It can be also used during sensitive construction procedures helping to the achievement of the desired results, with economic and safety benefits.

The new railway crossing over the River Sado, at Alcácer do Sal, consists in a large bridge with a complex structural system, built by innovative construction methods and in an environmentally sensitive area. These were the main reasons for the installation of a structural monitoring system in the bridge, but also acted as constraints for its development and implementation.

After a brief description of the bridge and the construction methods used, this paper presents the bridge structural monitoring system, including the different types of equipment used in order to get more accurate measurements. However, the focus of the paper is the experimental results obtained during some critical construction operations like deck incremental launching and arches' hoisting.

2. Bridge description

The new railway crossing over the River Sado has a total length of 2,7 km, including two approach viaducts and a main bridge, with three continuous spans of 160 m.

The main bridge is a bowstring arch bridge, with three continuous bowstrings with a single plane of hangers on the bridge axis (Fig. 1). There are 18 hangers in each span, 8 m apart, with solid circular sections of 200 mm diameter and steel grade S 355 NL, with a maximum length of 22,8 m.

The bridge deck has a steel-concrete trapezoidal composite section, 15,85 m wide, allowing the installation of two railway tracks.

Fig. 1: General view of the bridge over River Sado

The steel arches have a variable hexagonal cross-section, with the width increasing from 1,49 m to 3,20 m and the high decreasing from 2,40 m to 1,80 m towards the top. The thickness of these elements varies from 120 mm in the base section to 60 mm in the top.

The four tubular reinforced concrete bridge piers have hexagonal cross-sections and are founded on piles with 2.0 m diameter.

The approach viaducts have a steel-concrete composite plate girder decks with spans of 45 m and 37,5 m. Their piers, abutments and pile foundations are built in reinforced concrete.

A detailed description of the bridge can be found in [1].

3. Bridge construction and incremental launching

The main bridge construction involved the following major operations: the incremental launching of the deck, the hoisting of the arches, the connection between the hangers and the deck and, finally, the slab deck concreting.

The steel box-girder was prefabricated and transported to the construction site in 10 m long segments. The assembling of these segments was made using a platform made on the last four piers of the south viaduct, adjacent to the river. Three launches were carried out after the assembling of each complete span. To reduce the bending moments in the cantilever, a 20 m long launching nose was used and two temporary piers were built in each span, thus decreasing the maximum cantilever span to 54 m. For each span, three days were required to carry out the launching operations.

Fig. 2: Deck incremental launching

Fig. 2 shows the first launching operation. In this picture a temporary pier (left) and part of the platform (right) are also visible.

Fig. 3: Arch elevation

Steel arches were prefabricated in segments 8 m long. After launching the entire deck, these segments were assembled over the deck in three parts for each span: the central part having a length of 50 m and two side parts 56 m long each. For its elevation, temporary towers were built from the temporary piers, as an extension of these. The hoisting of each arch was performed in three steps. Firstly, the central part was hoisted in both ends to its position. Afterwards the south end of the north part was raised into position while the northern end was only free to rotate (Fig. 3). Finally, the same procedure was followed with the south part.

All three parts were lifted with the hangers connected to the arch at the top and with a sliding temporary bearing at the bottom, which allowed longitudinal movement during the lifting operations.

In order to minimize internal forces introduced into the arch, as a result of the erection scheme, imposed rotations at the joint sections were envisaged by raising the temporary bearings and allowing longitudinal movements, prior to each main welding of the three parts of the arch. To assemble the arch with the deck, the temporary piers in the river were lowered by approx. 100 mm. Once the correct rotation was achieved, the arch was welded to the deck sections [1].

After the welding of the arches to the deck, the hangers were adjusted and connected to the deck by the lower anchorages. For that purpose, the geometric control was very useful for setting the correct forces in the hangers.

A downward displacement of 1300 mm was imposed at the intermediate supports, after installing and adjusting all the hangers, in order to reduce negative permanent bending moments at deck continuity sections over theses supports [2].

Finally, the temporary supports were removed, the slab deck was concreted and the remaining works carried out.

4. General plan for structural monitoring

The bridge structural monitoring system includes the measurement of deck rotations and vertical displacements, strains and temperatures in both deck and arches, as well the joints movement, as presented in Fig. 4.

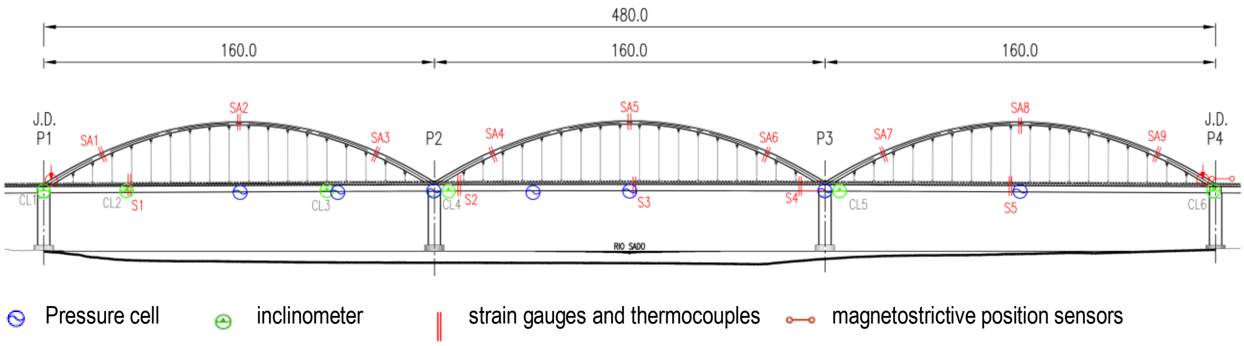

Fig. 4: Structural monitoring general plan

The measurement of vertical displacements is carried out through a hydrostatic levelling system associated to pressure cells. These cells are placed in the three mid-span sections of the bridge and in the quarter-span close P2, in order to measure the vertical displacement, and also at top of the piers P2 and P3, to serve as reference.

This hydrostatic levelling system and the magnetrostrictive position sensors used for measuring the joints movements were installed only at the end of construction.

The remaining devices were fixed and used during construction and are presented in more detail in the following sections: two-axis gravity referenced inclinometers to measure longitudinal and transversal rotations; PT100 placed across the thickness of the slab deck elements to obtain the thermal gradients; finally, strain gauges and NTC thermistors were used in five sections of the deck and in nine sections of the arches to measure strains and temperatures in steel elements (Fig. 4). Not mentioned in this paper were also mounted 25 vibrating-wire strain gauges, placed in five sections of the slab deck and several specimens for creep and shrinkage were made.

The control of all monitoring procedures in made from an industrial computer, installed on the deck over pier P2 (Fig. 5). The acquisition and real-time data validation are conducted by synchronous routines, as explained in [4]. The system is permanently available on-line, by means of a broadband cellular connection, and is capable of automatically validating data using statistical robust indicators and goodness-of-fit tests [4],[5]. Data storage is performed automatically using a MySQL server and an intermediate SFTP server, for greater internet security [5]. Data Fusion and real-time control of data are performed using Unsupervised Data Mining techniques such as Principal Component Analysis and Cluster Analysis, as described [6],[7].

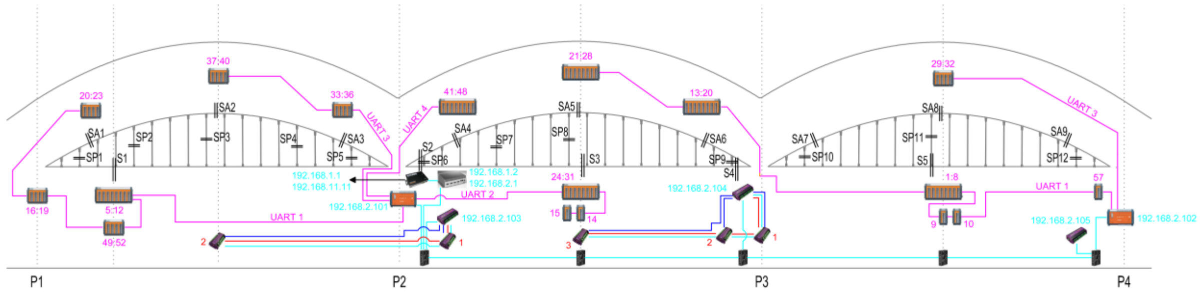

Fig. 5: Data acquisition system

5. Monitoring during deck incremental launching

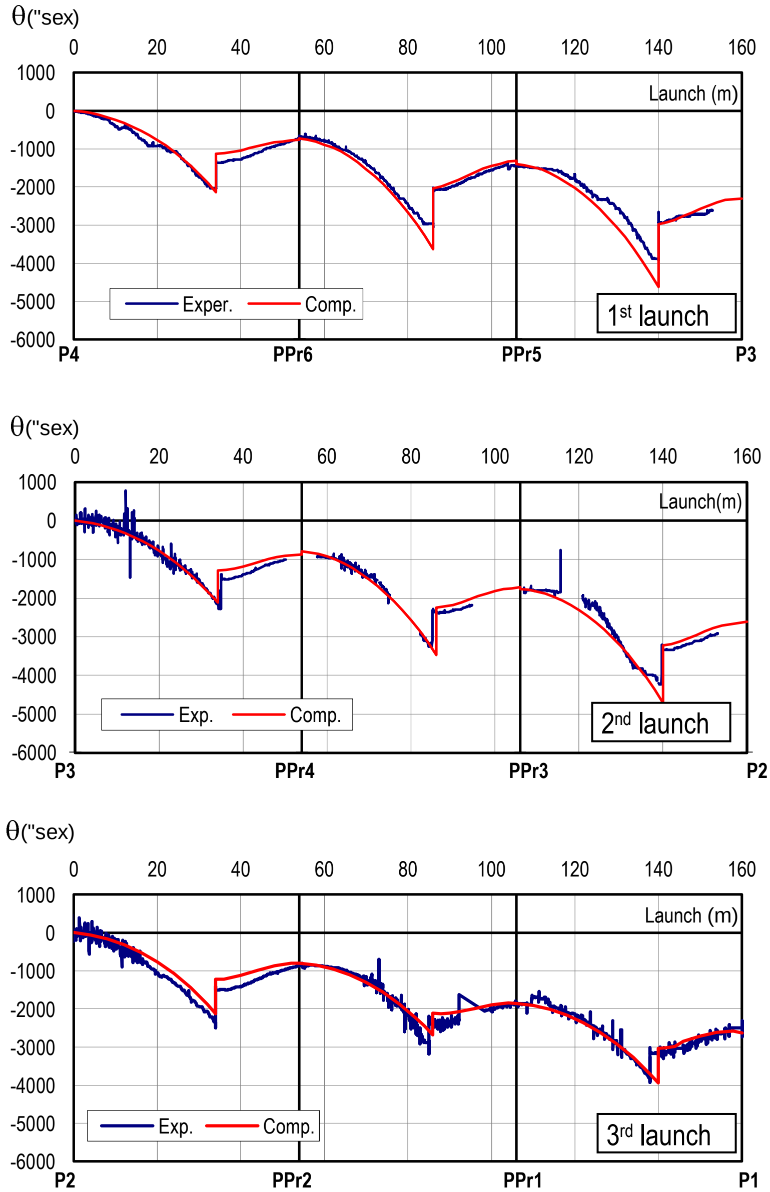

Fig. 6: Rotations at the cantilever free end

The structural behaviour of the deck during the incremental launching was simulated using a set of three dimensional numerical models developed in SAP2000 [3]. The definition of these models was made based on physical and geometrical characteristics defined in the design. This way was possible to predict the significant variations in the different measured values caused by the incremental launching.

The incremental launching of the deck was essential in the conception of the structural monitoring system, mainly, regarding the span P1-P2. In fact, section S1 (Fig. 4) is placed at the location of maximum bending moment during the incremental launching. This section was instrumented with strain gauges and two-axis gravity referenced inclinometers (another was placed at the cantilever free end).

Fig. 6 presents the rotations in the cantilever free end during the incremental launching.

A first remark is to the similarity between the values measured in the three operations: the alternated effects of the increasing cantilever span and of the temporary supports are obvious. An additional comment should be made about the values of these rotations. Indeed, the rotation at each launch was greater than 1 degree prior to the moment when the launching nose is supported by a definitive pier. In these cases it is clear the correlation between calculated and experimental values.

As previously mentioned section S1 was instrumented with seven strain gauges placed as presented in Fig. 7, in a similar way of the remaining sections of the deck (S2 to S5, see Fig. 4).

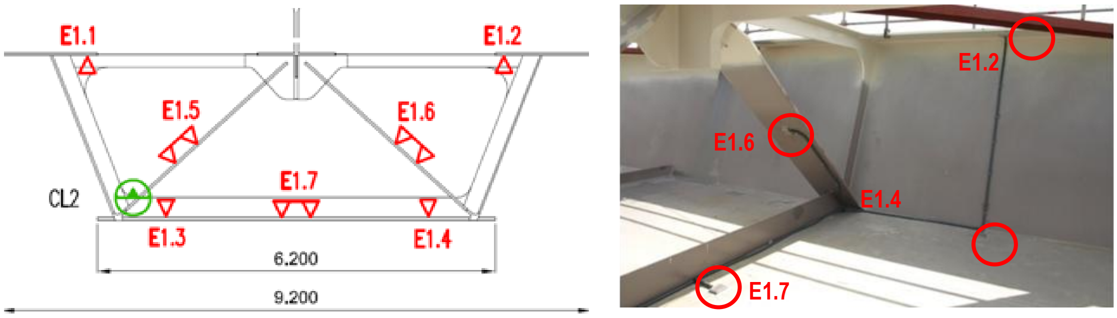

Fig. 7: Deck cross section S1: strain gauges location

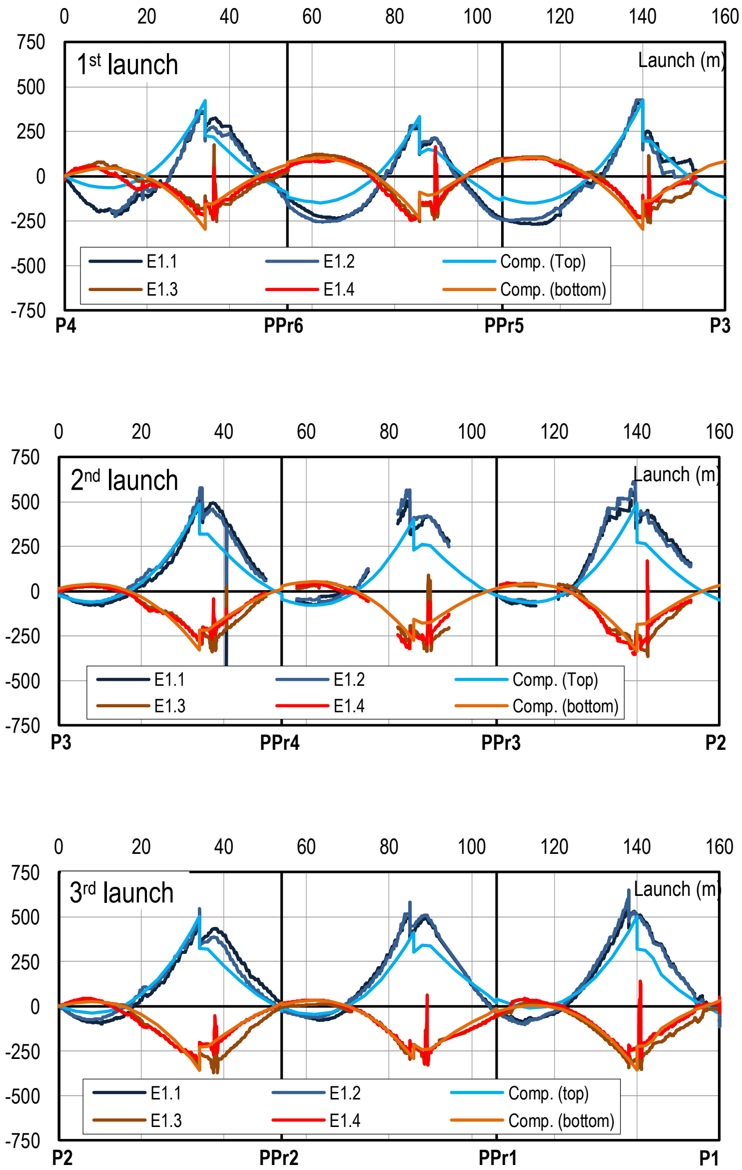

Fig. 8: Strains at section S1 during incremental launching

Top and bottom flanges strains measured at section S1 (Fig. 4) during the incremental launching are presented in Fig. 8 along with the numerical results.

This section endured large variations in bending moment. Actually, when the incremental launch started, this section was 34 m apart the pier P4 and was subject to a positive bending moment. This positive bending moment decreases as the section approaches the pier, reaching zero when the section was at a distance of 14 m from the support, and getting a maximum negative value with the maximum cantilever span; then, when the free end of the launching nose reaches a support, a signal inversion of the bending moment occurs.

An important added value of the monitoring carried out, is the knowledge that throughout the process of incremental launching the actual behavior corresponded to expectations (good correlation between measured and calculated values), not having been induced excessive stresses that could cause permanent deformation.

6. Monitoring during arches' hoisting

Table 1: operations for hoisting the arches

Operation	Description
1	Elevation of the central part of the arc
2	Elevation of the north part of the arc
3	Elevation of the south part of the arc
4	Imposed rotations at the joint sections
5	Welding of the three parts of the arch
6	Lowering of the temporary piers
7	Assembling the arch with the deck

The different operations involved in arches' hoisting are summarized in Table 1.This construction procedure leads to the instrumentation of the mid-span sections of each part of each arch with strain gauges and thermistors, as presented in Fig. 9.

The purpose of this option was to verify the efficiency of the construction process, as the imposed rotations and the lowering of temporary piers, in the elimination of bending moments installed on the arches during its hoisting.

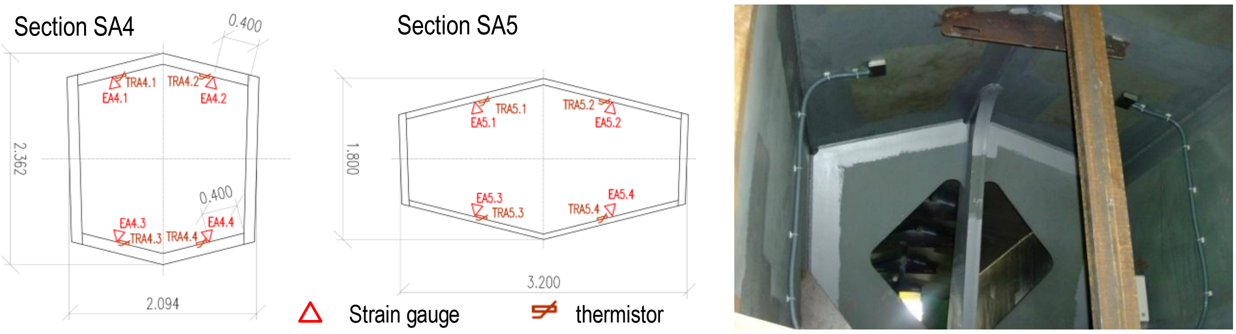

Fig. 9: Central arch: cross sections SA4 and SA5

The three arches had similar behaviour during their hoisting. Such behaviour is exemplified in Fig. 10, which presents the variation of longitudinal strains measured in the mid-span sections of the three parts of central arch (sections SA4 to SA6, see Fig. 4). In this figure, the numbers defined in Table 1 are used to identify each step of the hoisting arches.

In this figure, once again, there is a good correlation between measured and calculated values. However, the focus of the experimental values presented in this figure is the evidence of effectiveness of the process constructive in the dissipation of the bending moments due to hoisting of arches. Indeed, for these three sections there is a huge difference between the strains at the top and bottom flanges after the elevation of the three parts of the arch, which decrease after in each new phase of construction. After assembly, the arch was subjected to essentially axial efforts.

Strains variation measured between different operations were mainly due to temperature effects. To illustrate this effect, **Error! Reference source not found.** presents the strains measured during 24h along the temperature measure at the same time. The figure includes two charts: the first with values measured before assembling the arch and the deck; the second with values achieved after assembling.

As can be seen, in the first case a rise of 15ºC causes a shortening of 60×10^{-6}. After assembling an increase of 14ºC causes variations of strains greater than 120×10^{-6}. This suggests that after assembling the thermal effect is more significant, which explains the higher disturbance which occurs in the graphs of Fig. 10, after that operation.

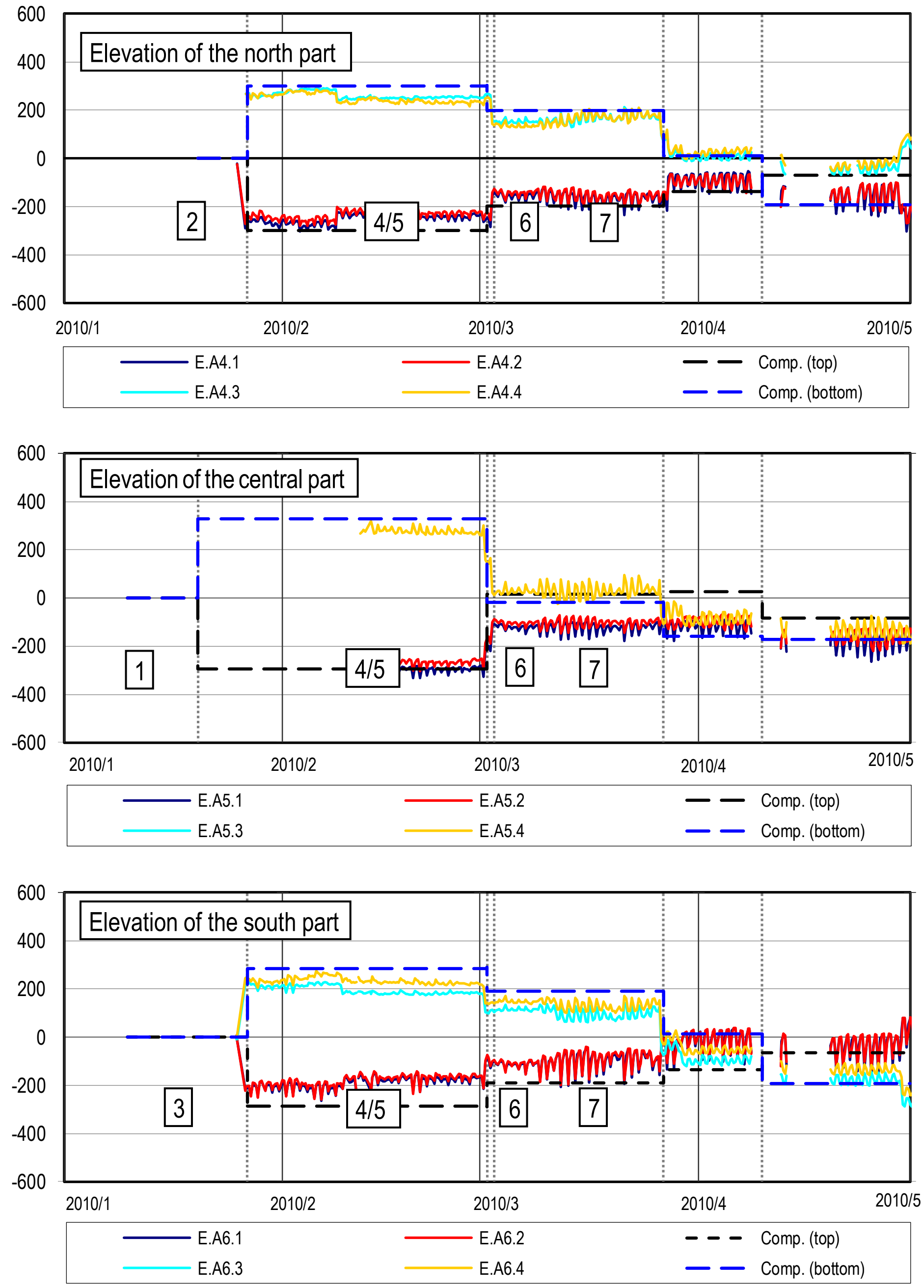

Fig. 10: Central arch elevation: experimental and computed strains

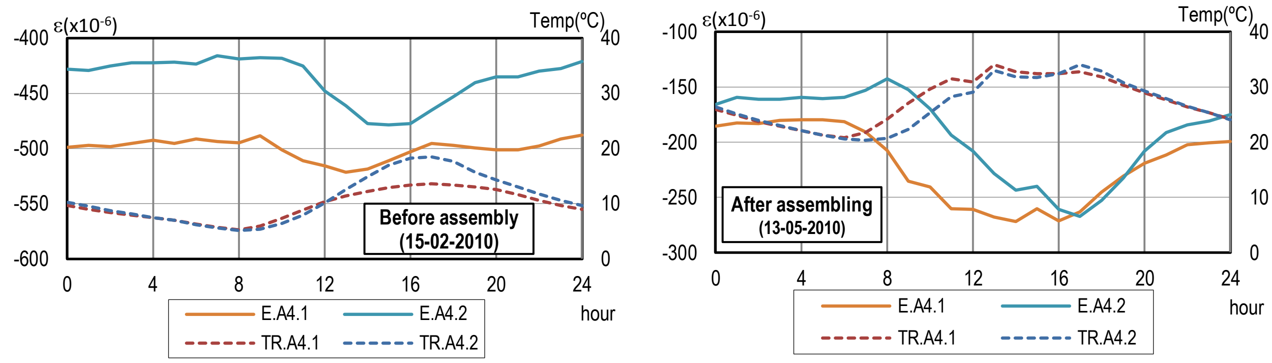

Fig. 11: Strains and temperature measured during 24h (section SA4)

7. Conclusions

The structural behaviour of the bridge over the River Sado was monitored during construction. The structural monitoring and data management system, both developed in LNEC, allowed access to experimental data in real-time as well as its comparison with data obtained from previously performed numerical simulations.

A main contribution of the used SHM strategy during construction was the ability to check in real time the performance and efficiency of the adopted construction methodologies.

Another important asset of the monitoring carried out, is the knowledge that throughout the process of launching, the actual behavior corresponded to expectations, not having been induced excessive stresses that could cause permanent deformation.

The use of such a structural monitoring system increased the reliability and enables early detection of any anomaly defect and its correction in time, avoiding higher maintenance costs.

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