Monitoring and testing of a new stone masonry arch bridge in Vila Fria, Portugal

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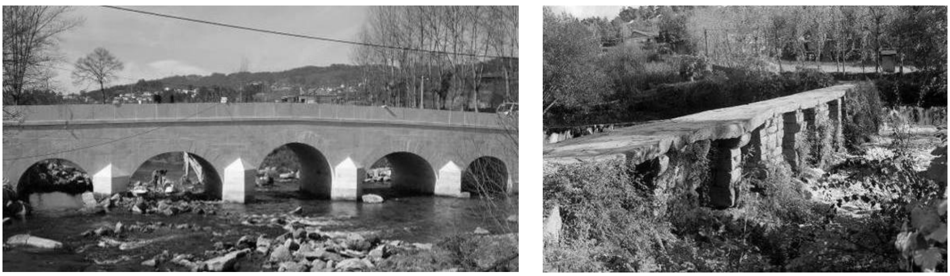
ABSTRACT: This paper reports on the monitoring and load testing of a new stone masonry arch bridge recently built over the river Vizela in Vila Fria, Felgueiras, located about 60 km from Porto, Portugal. Besides highlighting the scientific purposes subjacent to this bridge construction initiative, the paper addresses basic aspects on the installed instrumentation and provides a general overview of sensor types, location and measured signals, as well as data acquisition and transmission systems. Some of the most relevant result measurements obtained from the load test and their comparison with available numerical analyses are also presented.

1 INTRODUCTION

The structural behaviour of stone masonry bridges still persists a challenging and pertinent topic of research, although it typically refers to old structures. The very fact that a large number of such constructions exist, in Portugal as in many other countries, under service conditions for which they were not designed, increases the problem relevance, Costa (2002). The need of improved knowledge based on the actual behaviour of real structures becomes even more stringent because some of such constructions exhibit clear signs of degradation and lack of maintenance that, quite often, suggest serious doubts on the safety level they can ensure under service conditions.

The case described in this paper is believed to assume particular relevance for the scientific and technical community interested in the field of structural analysis and monitoring of old masonry constructions, bridges in particular. The case study refers to the new stone masonry bridge Fig. 1a in Vila Fria - Portugal, over the Vizela River, recently built according to traditional techniques of masonry construction to replace an old and very deficient passage Fig. 1b.



a) b)

Figure 1 : Vila Fria Bridge. a) new bridge and b) old passage.

The new bridge consists of a 60m long and 6m wide two-ramp deck, supported by five arches with 4.8m to 6m span, four piers and two abutments; the design and construction details are addressed in a companion paper Costa *et al*. (2007). The multidisciplinary work concerned with the design, construction and structural behaviour study of the bridge has involved several components, namely: structural modelling and analysis, laboratory and *in-situ* testing for numerical model calibration, instrumentation and measurement of structural and material parameters, *in-situ* data acquisition and remote monitoring. Undoubtedly, amongst all these interesting issues, the greatest motivation for this initiative was the possibility of continuously monitoring this construction from its very beginning.

All the instrumentation and data acquisition equipments are now installed and working since almost one year. Data collected can be accessed locally or remotely in a dedicated server existing at FEUP where all the information is stored in an adequate database that feeds a specific internet site. Meanwhile, a load test was performed on the bridge and some of the gathered results are briefly addressed herein, aiming at comparison with numerical results.

2 MONITORING OBJECTIVES

The monitoring campaign of the Vila Fria bridge was motivated by a previous work that consisted in the structural modelling and analysis of a similar case, Costa (2002), for which several difficulties were found concerning the calibration of material and structural parameters, the confirmation of the obtained numerical results and the influence of some issues such as the infill and spandrel wall stiffness.

In this context, the new bridge instrumentation should provide measurements of: *i)* global displacements of selected points of the structure, namely in arch crowns; *ii)* relative displacements in joints between stone blocks, namely opening/closing and slipping; *iii) r*elative displacements between opposite spandrel walls; *iv)* strain and stress values in some arch blocks; *v)* vertical stresses in the filling material and *vi)* temperature.

Through such measurements, one basic scope is to assess the numerically obtained deformed shape of the structure (both at the global and local levels) by comparing it with the real one under controlled loads. The response of strain measurements (and the corresponding elastic stress calculation in stone blocks using experimentally obtained deformability parameters) and infill pressures are expected to allow clarifying the stress path and distributions in the arches, infills and piers, and to assess the corresponding numerical results. Finally, temperature measurements in several bridge zones, particularly close to other sensors, are essentially devoted to allow corrections of raw data to account for temperature effects.

3 INSTRUMENTATION AND DATA ACQUISITION

By taking profit of the construction process a large instrumentation network was installed in the bridge, including both electrical type and fibre optic based sensors, the later particularly relying on fibre Bragg gratings, Ferreira *et al* (2004). Accordingly, two different data acquisition systems were adopted, one for the electrical signals and other specific for optical signals. The basics of the installed sensors are described in the following paragraphs, according to their types and to the measurement they can provide; brief topics are also included for data acquisition and transmission systems.

3.1 Global displacements

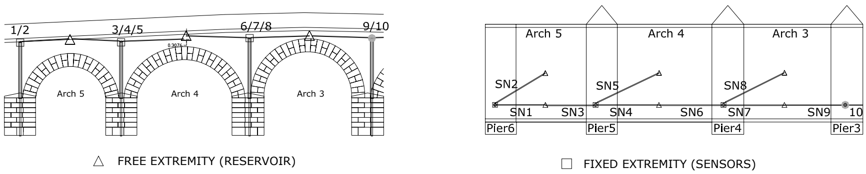
The continuous measurement of absolute movements such as the global displacements of a bridge is always a matter of great importance but is also usually a very tricky task, due to the difficulties on having adequate reference systems. Therefore, in order to partially overcome this problem, several ultra-low differential electrical pressure sensors were adopted to measure differential vertical displacements between a series of points along the Vila Fria bridge deck. The basic principle consists on measuring the pressure variations in a given fluid existing in a pipe that connects the sensor to a fluid reservoir at atmospheric pressure, as shown in Fig. 2.

Any change in the vertical level of the sensor or the reservoir enforces a fluid pressure variation that is read by the sensor and transformed into an electric signal proportional to the differential level change. For this reason, these pressure sensors are herein named as level sensors.

a) b) c)

Figure 2 : Level sensors: (a) sensor, pipe and reservoir, (b) sensor box and (c) reservoir in the bridge.

The sensors and respective reservoirs are installed along the bridge according to the layout illustrated in Fig. 3 for half-bridge. Since it was not possible have a fixed point outside the bridge to serve as reference point for vertical displacements, the system was prepared to provide differential displacements between the arch's mid-span and their respective piers. Accordingly, fifteen sensors were placed inside the bridge over the piers, more precisely on concrete columns fixed to the upper face of each pier adjacent to the bridge downstream spandrel wall. Sensor reservoirs were placed on similar columns over each arch mid-span section, both along the bridge longitudinal axis and along the alignment of the sensors, at adequate height levels compatible with the measuring range of the sensors. These columns are isolated from the bridge fill by means of larger diameter concrete tubes in order to avoid any interference of fill movements on the sensor response.



a) b)

Figure 3 : Level sensors: (a) elevation layout, (b) plan layout.

Sensors were adopted from Honeywell, type 160PC, adequate for a pressure range of 0 to 0.254 m of water column; together with 12 bit data acquisition cards, the system provides a global resolution below 0.1mm for displacements that is enough for the expected movements. Besides some preliminary laboratory calibrations, the final calibration procedure was undertaken after installation in the bridge using a specially designed device to allow moving the reservoir up and down, under controlled displacement conditions while recording both the displacement and the sensor signal. Typical displacement/signal relations were obtained as d(mm)=31*V(Volt) that is approximately consistent with the factory sensor specifications.

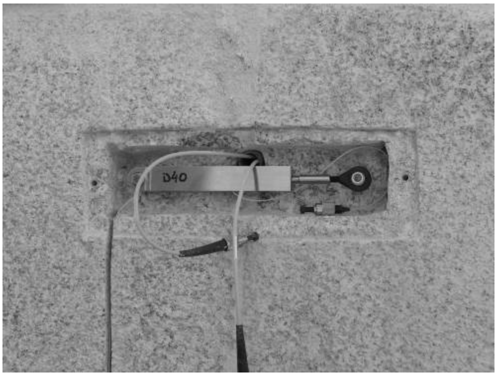
For the compensation of temperature effects in the whole system (sensor, fluid, pipe and reservoir), a dummy sensor was adopted, with similar pipe length as for the other sensors and with the reservoir attached to the same support device as the sensor itself. Therefore, since level changes between the sensor and the reservoir are not possible, the sole cause of fluid level variation (and corresponding sensor signal) is the change of temperature that is also recorded by means of appropriate sensors.

3.2 Relative displacements in joints

In order to measure relative displacements of opening/closing and sliding between adjacent stone blocks under service conditions, displacement transducers were installed on one of the joints of each arch keystone (near the mid-span sections) following a "Z" configuration. Additionally, in joints near the ¼ arch span sections the same type of transducers were also installed, although only in three arches, the central one and the two adjacent others. Fig. 4a shows the general transducer layout for one of the arches whereas Fig. 4b shows one of the transducers already duly placed in one of the cavities on purpose opened in the downstream bridge façade. This transducer type, called LPDS (Linear Position and Displacement Sensor), is based on optical fibre Bragg gratings and was specifically developed and designed for this monitoring application. Details about the sensor and working principle can be found elsewhere, Ferreira *et al.* (2006).

In total, 49 LPDS were distributed by several joints in the downstream façade and in the intrados of some arches. The later are mainly devoted to measure transverse expansion movements of the arches and joint opening in the longitudinal direction along the bridge axis where the structure is more flexible concerning vertical deflections. Fig. 5a shows the schematic layout of some of such sensors, placed in an arch intrados.


a)

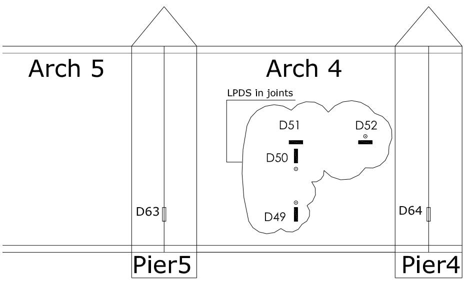


b)

Figure 4 : Displacement transducers in the joints: a) general layout for one arch and b) layout in a joint

3.3 Relative displacements between opposite spandrel walls

The lateral pressure effects of the infill on the spandrel walls, are monitored by transversal long gages installed between opposite zones of those walls, namely in one of the abutments and above three piers. Fig. 5a shows also the schematic layout (plan view) of two such sensors above piers P4 and P5.



a)

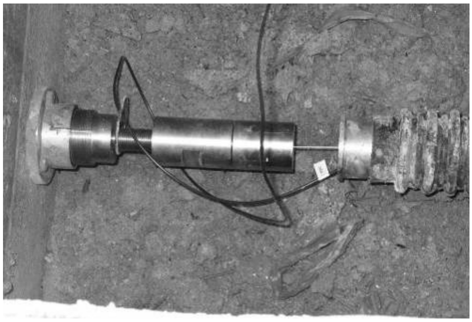

b)

Figure 5 : Optical displacement transducers: a) layout in arch intrados and between spandrel walls; b) transversal long gage.

As for LPDS, these sensors (four in total) are also based on optical fibre Bragg gratings, Ferreira *et al*. (2006), and the desired displacement is obtained by measuring the axial strain of an invar wire attached to each spandrel wall with an initial pre-stress and by multiplying this strain by the wire length. Fig. 5b shows one fixing zone of one such sensor next to the spandrel wall, where it can be seen the fixing element, the tensioning device, the invar wire and the extremity of the waterproof stainless steel tube protection the setup. Externally, the whole system is involved by another steel tube with significantly larger diameter than the first one to protect it from filling material settlements.

3.4 Strain in stone blocks

Several arch stone blocks were also provided with strain gauges in intrados and extrados surfaces in order to measure local deformation. Subsequently, with previous knowledge of the Young modulus and Poisson ratio of the stone it is possible to calculate the corresponding stress in the linear elastic domain which is quite acceptable for this type of constructions that normally exhibit very low stress values. Measurement of strains in the intrados and the extrados thus aims at assessing the strain and stress distribution in the arch thickness. Therefore, pairs of strain gauges were glued in the longitudinal and transversal direction of the bridge, in stone blocks of two half-arches (A3 and A4) adjacent to the pier P4 (Fig. 6), along two longitudinal alignments. Strain gauges were mainly of electrical type although some of optical type were also installed, accounting for a total of 48 points where strain measurements are taken.

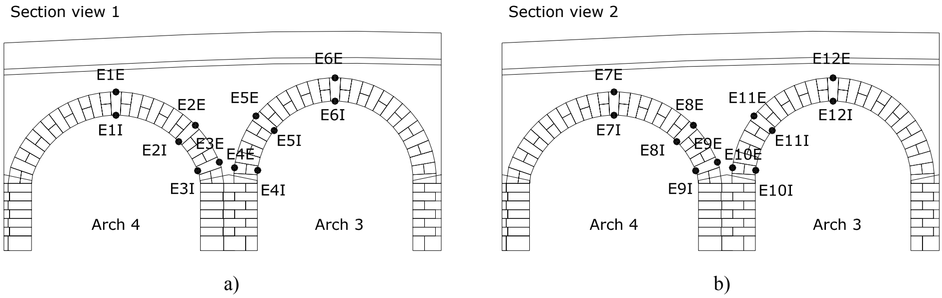


Figure 6 : Strain gauge layout. a) Downstream façade alignment and b) central alignment

3.5 Infill pressure and arch stresses

Vertical pressure in the infill is measured at two levels above pier P4 in between arches A4 and A3, by recourse to a pair of total pressure cells as commonly used in geotechnical works. These two cells aim at evaluating the load dispersal inside the most deep infill zone. Therefore, one such cell (C1) is placed on the interface between the pier P4 and the infill, whereas the other cell (C2) is placed right above the previous one at the same level as the extrados face of the arch A4 keystone. In addition, five other cells (C3 to C7) were also adopted to measure the normal stress in several points of the extrados and intrados of the arch ring along the longitudinal axes of the bridge, aiming at evaluating the stress path and distribution along the ring.

Fig. 7a shows the location of these cells, whilst Fig. 7b illustrates the circular cell C1 already placed on the top pier face and Fig. 7c shows the operation of placing the half-moon cell (C7) in a slot previously opened in the keystone of arch A4. These cells were provided by Geokon and refer to the models 3510 and 3500: cells C1 and C2 range between 0 and 600 kPa while cells C3 to C7 may read up to 1000 kPa. However the working principle is the same and consists of a given fluid inside the closed space between the two cell plates; when these plates are compressed, the fluid inside reflects the corresponding pressure change that is related to the normal stress exerted on the cell. A pressure transducer connected to the cell allows reading the pressure change that is sent to the data acquisition system as an electric signal

a) b) c)

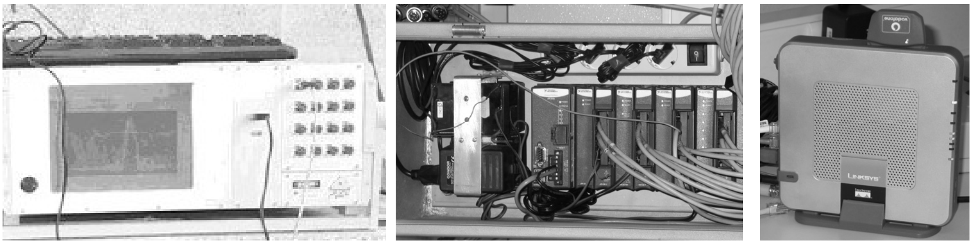
Figure 7 : Total pressure cells. (a) Location, (b) circular cell (pier-infill) and (c) half-moon cell (inside stone blocks).

3.6 Temperature

Temperature is measured in several points of the bridge, namely in the downstream façade (where displacement transducers are located), in the intrados and below the pavement near the level sensors. The basic objective is to provide measurements for correction of the remaining sensor response due to temperature variation effects. Attached to each pressure cell there is a thermistor, a sort of temperature sensor consisting of a resistor that varies the resistance value. In addition, several other (28 in total) optical type temperature sensors were also distributed along the bridge in strategically chosen locations within the optical fibre network.

3.7 Data acquisition systems

Data of all sensors are collected by appropriate acquisition systems (see Fig. 8), namely one BraggMeter measurement unit (Fig. 8a) containing an optical switch from FiberSensing and one Compact Fieldpoint acquisition unit (Fig. 8b) from National Instruments, Arêde *et al*. (2006). Both equipments are connected by TCP/IP to a router (Fig. 8c) allowing wireless data transmission by GPRS/UTMS to a remote server at FEUP where monitoring results are collected in a specifically designed database and accessible through an internet site.



a) b) c)

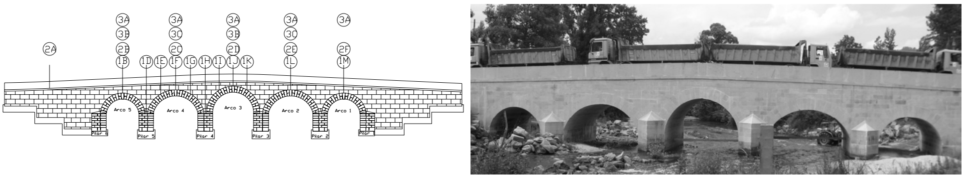
Figure 8 : Data acquisition and transmission (a) BraggMeter optical unit, (b) Compact Fieldpoint unit and (c) GPRS/UTMS wireless router and modem.

4 LOAD TEST RESULTS AND COMPARISON WITH NUMERICAL ANALYSIS

After completion of all the activities related to instrumentation, calibration and whole system testing, the load test on the bridge was carried out using four trucks loaded to their maximum capacity around 39 ton (40% for the two front axles and 60% for the two rear axles). For testing purposes, this value fits well with the 30 ton vehicle load prescribed by the Portuguese National Code Standard for Safety and Actions, RSA (1985), for class II bridges according to which the new Vila Fria was designed.

Three vehicle arrangements were adopted to obtain the most unfavourable structural effects, Costa (2007). Particularly, in order to achieve the most concentrated load as possible, the arrangement 1 consisted in just one pair of side-by-side trucks in twelve stopping positions along the whole bridge length defined by the middle position of the two rear axles.

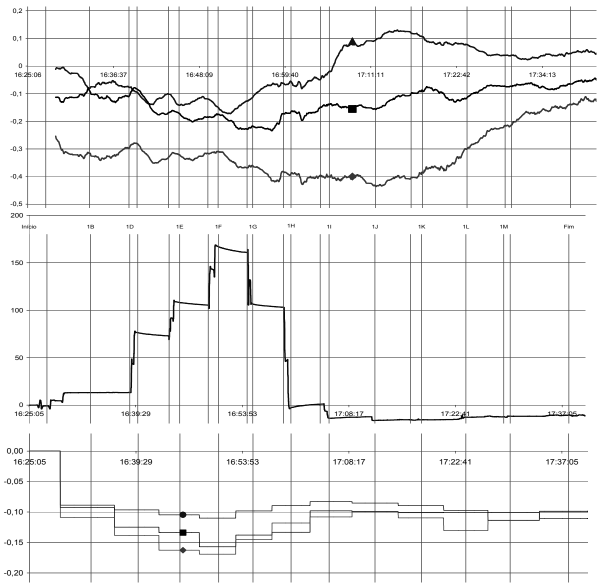
Fig. 9 shows the stopping positions for the trucks along the bridge for the three arrangements (1A to 1M, for the first; 2A to 2F for the second; 3A to 3C for the third), as well as a photo of one test stage (3A, with trucks all over the full length of bridge arches).



a) b)

Figure 9 : Load test. (a) Truck stopping positions and (b) truck arrangement for position 3A.

Although not corresponding to the larger total load, the arrangement 1 was found to produce the most unfavourable effects. Therefore, results herein addressed refer solely to that load configuration and particular attention is given to the arch 4, the most instrumented one and likely to exhibit more pronounced effects. Moreover, due to paper length limitations, results as shown in Fig. 10 only focus on the vertical displacement (Fig. 10a), compressive stress (Fig. 10b) and joint opening/closing (Fig. 10c) in that arch crown. Vertical alignments of these plots correspond to the several stopping positions and are identified in Fig. 10b.



a) Vertical displacements of the arch crown relative to its piers. The plot with lozenge marks refers to the sensor near the bridge axis. Negative values refer to downwards movement.

b) Stress evolution (kPa) in load cell C7, for all positions of the first vehicle arrangement. Note the most unfavourable situation for position 1F right above the arch crown, as expected. Positive values refer to increase of compression.

c) Joint normal displacements near the arch crown. Plots with lozenge marks and with square marks refer to the intrados and extrados sensors, respectively. Note that only closing (-) is observed.

Figure 10: Load test results (a), (b) and (c) for arch 4 and load arrangement 1.

By the time this paper was written, long term monitoring results were not yet processed and analysed for which they will be published later elsewhere.

Numerical simulations were performed to simulate the response of the load test, using the general purpose computer code CAST3M, Pasquet (2003), and finite element modelling strategy as reported in a companion paper, Costa *et al.* (2007). As stated there, this strategy still corresponds to linear elastic analysis (though involving individualized joint elements with different stiffness from the blocks) while the experimental results of joint parameters are not yet fully available. For comparative purposes, the main results concerning arch 4 effects, namely deflections, joint movements and keystone normal stresses are summarized in Table 1, as obtained due to load test actions and for the position 1F (trucks with rear axle load centred with the crown of arch 4 in the bridge longitudinal axis).

Table 1 : Numerical simulation results exclusively due to load test truck actions, position 1F

Effect	Location	Model result	Measured result
Vertical displacement (mm)	Crown of arch 4 – bride axis	0.41	0.31
Normal stress (kPa)	Crown of arch 4 – bride axis	157	170
Joint closure (mm)	Crown of arch 4 – downstream face	0.06	0.16

Despite this synthetic manner, the above described results provide a first insight into a quite reasonable agreement of results between experimental measurements and numerical calculations, at least for these most representative effects. However, new calculations are still to be undertaken with more substantiated material properties that will allow more in depth comparisons between measurements and numerical analysis, in order to find out the extent of this result agreement.

5 CONCLUSIONS

The previous sections provided a general overview of the most important issues related with the instrumentation and monitoring of the new stone masonry arch bridge recently built in Vila Fria, Portugal.

As clearly evidenced in the above described, this quite unusual initiative has involved a large number of multidisciplinary tasks, not always easy to make compatible, that, for obvious limitations of paper length were not possible to described to the desired extension herein and in a companion paper focusing on the bridge design, construction and other issues, Costa *et al.* (2007). However, particularly in this paper, it was possible to show the basics of the adopted instrumentation and the type of problems arising for a proper installation *in-situ*. Several mechanical and physical quantities can be measured with the implemented sensor network and, by means of the corresponding data acquisition and transmission systems, the results can be accessed from everywhere in a dedicated website (http://remotelese.fe.up.pt/).

Based on the results of the bridge load test, it was already possible to check that significant part of the adopted instrumentation is working properly and giving results in good agreement with numerical analysis performed so far. Although these simulations are still in an upgrading process, due to material parameters that are still being experimentally evaluated, the good comparison of results with the measured data is really encouraging as it contributes for confirming the suitability of the instrumentation for the desired purposes.

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