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Structural monitoring using fiber optic sensors of a pre-stressed concrete viaduct during construction phases

Giuseppina Uva [a,1], Francesco Porco [a,2], Andrea Fiore [a,*,3], Giacinto Porco [b,2]

[a] DICATECh, Politecnico di Bari, via Orabona 4, 70126 Bari, Italy
[b] Dipartimento di Ingegneria Civile, Università degli Studi della Calabria, via Ponte Pietro Bucci, 87036 Rende, Cosenza, Italy

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ABSTRACT

The paper presents a study about the monitoring of a pre-stressed reinforced concrete viaduct in Bari (Italy), by means of an optical fiber system embedded into the structural elements. The application case had two objectives: controlling the structural efficiency during the phases of construction, and allowing, in the future, the periodical check of the structural performance under service loads. Sensors were directly anchored to the prestressing strands during the manufacturing phases of the precast beams. By processing and analyzing the data acquired by the system during the different construction phases, it was possible to assess the strain variations related to load increments and stress losses, by comparing them with expected theoretical values. The specific case study shows that the availability of real-time monitoring procedures is nowadays a precious tool for checking the structural safety of critical facilities, in particular bridges.

1. Introduction

In the last few years, the safety assessment of existing RC structures and infrastructures has assumed a great relevance: in Italy the problem is particularly important, also from a “quantitative” point of view, considering that a large part of the Italian building and infrastructure stock is dated back to the 60’s–80’s and is presently affected by a severe level of damage and degradation of materials. RC structures built in this period, actually, are particularly vulnerable to environmental actions, mainly because they were built in the absence of specific constructive detailing aimed at providing the quality of “durability” [1]. The “*evident deterioration of the mechanical characteristics of the materials*” is indeed one of the situations in which the current Italian Code [2] requires to perform a specific structural assessment. Often, the degradation of materials or the loss of the structural integrity are non-immediately visible, requiring expensive and invasive protocols of inspection that should thence become an integral part of the regular maintenance programs. In other cases, the evidence of the degradation (which can involve carbonation, steel corrosion, chloride attacks, spalling and debonding of concrete) appears when the damage is very extended on the structural elements. The consequence is often that the necessary actions are not taken at the right time and the retrofitting interventions become much more complex and expensive, involving in some cases even the demolition/reconstruction option.

* Corresponding author. Tel.: +39 080 5963832; fax: +39 080 5963832.
E-mail address: andrea.fiore@poliba.it (A. Fiore).
[1] Associate Professor.
[2] Researcher.
[3] PhD Student.

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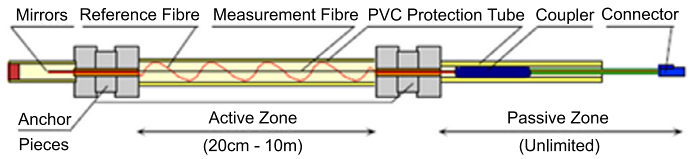

Fig. 1. Scheme of a standard SOFO sensor.

In this regard, local administrations are presently facing a very serious problem about the systematic vulnerability assessment and the decision about the proper actions to be taken.

The experience is pointing out that the assessment process, and in particular the "knowledge protocol" for the achievement of an adequate level of knowledge [2,3] is a complex matter, and requires the allocation of relevant economic and technical resources.

In the case of strategic facilities, it is evident that the early detection of structural failures or degradation of materials is crucial in order to allow the prompt application of preventive measures and to guarantee an effective management of the maintenance programs. Indeed, an increasing attention has been devoted to these issues by the scientific and technical community and also by all the most recent technical standards [2–5]. When dealing with lifelines or strategic buildings (facilities for the civil protection; bridges; hospitals, ...), a high social interest is also involved, and the regular control becomes crucial also in view of the safety verification after exceptional events (earthquakes, hydro-geological instabilities, ...).

A useful support is offered by the modern systems of structural health monitoring which, if properly designed, allow to perform a real-time control of the structural integrity, and represent a precious complement for the related decision process (choice, scheduling and implementation of routine/special maintenance programs).

In recent years, the field of the structural monitoring has been invested by a great technological development, which has involved, besides, a widespread use of fiber optic sensors [6]. These devices, which are dielectric materials unaffected by electromagnetic disturbance, represent an ideal choice for many applications, are easy to handle and are able to record even very small lengths (up to 2 micron) with a high precision. There is a great variety of optic sensors, which are used both for industrial applications and for the structural control of civil structures [7–13]. The several research recently published show the interest from the scientific community to the issue of structural health monitoring by means of last generation sensors [14–16].

In the application case which is discussed in the paper (a viaduct recently built in the city of Bari, Italy), the severe hydro-geological vulnerability of the site suggested the installation of a structural health monitoring system, which was based on the "SOFO" technology. This kind of fiber-optic system belongs to the category of "long base" sensors, which are designed for acquiring data on a long base of measure (a few meters).

In the following sections, after providing a short overview of SOFO monitoring systems for bridges (Section 2), the proposed procedure is described with reference to the case study (Section 3), showing its main features and discussing the results obtained (Sections 4 and 5).

2. Application of SOFO technology to bridge monitoring

Over the past 20 years, a large number of applications of optic fiber sensors have been developed for the monitoring of great infrastructures, both new and existing [17]. With regard to new prestressed RC structures, these monitoring systems allow – for example – to appraise the expected stress loss during the construction phases and the service life [18–20]. In the phase of concrete casting, they are also very useful in order to evaluate the effect of the deformations related to the shrinkage, and afterwards, to monitor the response of the new structure under growing loads [21]. The presence of the resident system, moreover, is a fundamental support for the structural vulnerability assessment of the structure after a number of years of service [22].

The SOFO (*Surveillance d'Ouvrages par Fibres Optiques*) sensor is made by a pair of single-mode fibers installed into a small protection tube ("Active Zone"), as shown in Fig. 1. The two fibers are called, respectively: "*measurement fiber*" and "*reference fiber*".

The *measurement fiber* is fixed over the element and follows its deformation. The *reference fiber*, instead, which is longer, is free to move within the duct. In order to record the deformations both under tension and compression, the measurement fiber is pre-tensioned (0.5% of the initial length). The measurement technique is based on the difference of length between these two fibers, which is only related to the mechanical deformation, whereas other perturbations (such as the temperature variation) affect the two fiber in an identical way, and do not influence the relative length difference [23,24].

The optical system is composed by commercial sensors. The SOFO measurement system is based on low coherence interferometry in single-mode optical fibres. The three main components of the system are a reading unit, the fibre optic sensors and the appropriate software. Its functional principle is represented in Fig. 2.

In order to measure the difference of length, a double low-coherence Michelson interferometer is implemented. The first interferometer is represented by the two above mentioned fibers embedded in the structure (*Active Zone*), whereas the second is contained in an external unit (*Portable Reading Unit*, Fig. 3) and is connected to the first by means of a cable (*Passive Zone*) which can be up to 5 km long. The portable reading unit is waterproof and battery powered, in order to

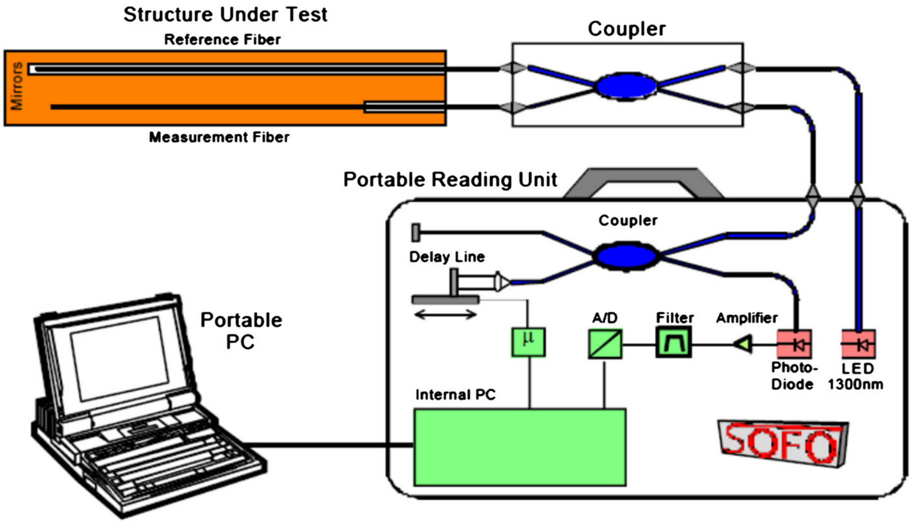

Fig. 2. SOFO system architecture.

Fig. 3. The external Portable Reading Unit.

allow the permanent installation even in humid environments. Data acquisition is performed at constant intervals of 7 s and results are automatically transferred, recorded and processed by a specific management software installed on a remote laptop computer. The measurement base of the standard sensor is between 20 cm and 10 m long, with a resolution of 2 μm (2/1000 mm) and an accuracy tolerance limit of 0.2% of the deformation. The dynamic range of measurement ranges from −0.5% (in contraction) to +1.0% (in elongation).

3. The case study

3.1. General description of the structure

The presented case study concerns a viaduct recently built in the city of Bari, Southern Italy (Fig. 4), which has 5 bays (named with the capital letters "A"–"E" in Fig. 5) each 22.30 m long, for a total length of about 112 m.

The deck, which is 16.40 m wide (with a 9.00 m wide roadway), is supported by 9 pre-stressed reinforced concrete beams spaced 1.525 m. The "I" beams are 1.25 m high and are connected by a continuous concrete slab 25 cm thick. The deck is completed by 3 transoms, two at the end of the deck (having a 40 cm height) and one in the middle of the deck (having a 30 cm height). The precast beams, supplied by a specialized manufacturer, are pre-stressed by stabilized straight strands. With regard to the constraints, the bridge is simply supported (the longitudinal constraint is fixed at one end of the bridge and mobile at the other). The lateral sliding determined by seismic actions or by creep is absorbed by specific dilation joints. The piers have a rectangular section (1.60 m$\times \approx$12.50 m) and are 14.51 m high. The upper part of the generic pier,

Fig. 4. View of the completed viaduct.

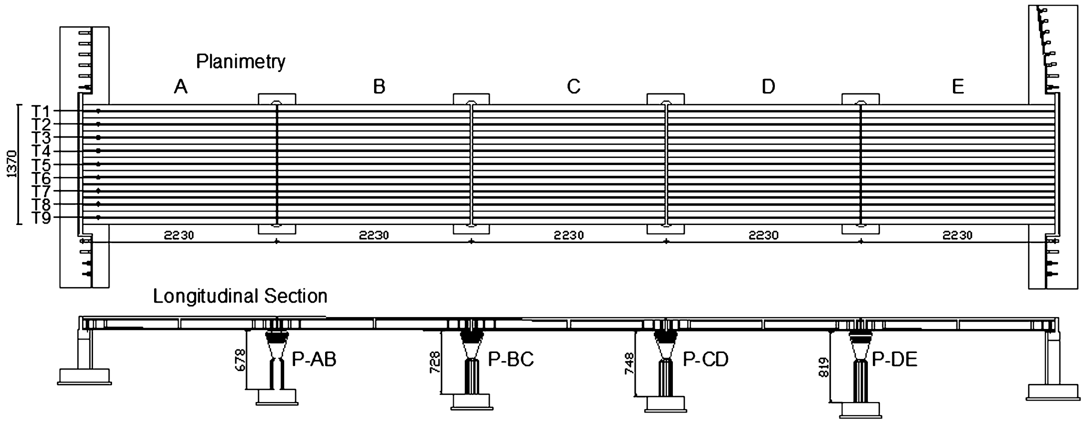

Fig. 5. Plan view and prospect view of the viaduct.

at the conjunction with the deck, is tapered from a minimum width of 1.00 m to a maximum of 2.70 m. Eighteen support devices (PTFE/steel bearings) are located on the top plane of the pier cap, which is 2.20 m × 14.10 m large.

3.2. The objectives of the monitoring system

The implementation of the monitoring system has the following objectives:

- OBJ 1 Performing a detailed investigation about the actual response of the precast beams at the different construction stages, with a particular attention to the stress losses in the pre-tensioned steel.
- OBJ 2 Assessing the overall structural response of the bridge during the different construction stages, by controlling the deformations in the deck and in the piers.
- OBJ 3 Supporting the technician responsible for the final inspection and acceptance tests on the completed structure.
- OBJ 4 Periodically assessing the structural performance and functionality during the service life.

The 1st objective is specifically focused on the control of the pretensioning effects in the beams, and is aimed at the detailed assessment of the theoretical behaviour of the prestressed beams. In fact, when the pretensioning method is used – as in this case, the monitoring system allows to quantify the actual stress losses in the strands, thanks to the acquisition of data before, during and after the concrete casting. It is possible, thence, to perform a systematic appraisal of the correspondence between the actual stress losses and the theoretical prediction provided by existing formulations (as proposed by the current technical standards and by the established scientific literature). This specific issue will be addressed in a forthcoming paper.

The 2nd objective is more general, being aimed at the evaluation of the global performance of the different structural elements during the construction stages. In particular, considering the specific structural type (simply supported bridge, with no structural continuity between the adjacent deck spans), the response of the deck and of the piers is uncoupled, and can be assessed independently. In this respect, the paper is focused on the deck (whereas the discussion of the assessment of the piers will be the subject of a forthcoming paper), as described in detail in Section 4.1.

Objective 3 is about the final test and inspection, necessary to confirm the proper completion of the construction. In this phase, the technician in charge usually performs a number of loading tests and measurements aimed at appraising the

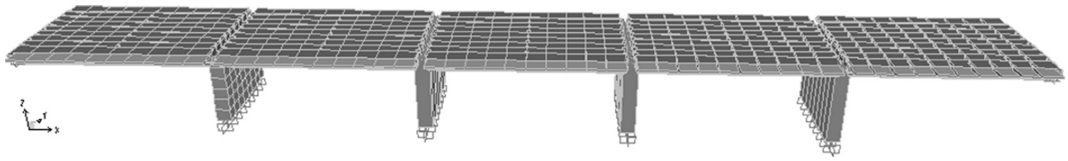

Fig. 6. The three-dimensional numerical model of the bridge.

effects of test-loads on the structural elements. The presence of the resident monitoring system allows to immediately derive some significant strain measures which can substitute or supplement the traditional instruments. This phase (Objective 3) is not here discussed but will be the subject of a forthcoming paper.

Objective 4 is the true real-time monitoring of the Viaduct during its service life: the systematic data acquisition performed by the monitoring system will become an integral part of the maintenance plan, capable to point out potential anomalies in the structural response, especially after exceptional events (earthquakes, floods ...). With regard to this aspect, in the paper a general methodology is defined, whereas the systematic application will be performed as long as the data will be available.

3.3. Permanent monitoring of the structural elements

In order to select the structural elements to be monitored and prepare the monitoring plan, a two-phase preliminary analysis has been performed.

In a first phase, some simple geometric considerations have been made in order to identify the most sensitive beams, by considering the following aspects:

1) Length of the spans.
2) Shape, dimensions and distance between the beams.
3) Number and position of the transoms.
4) Dimensions and position of the roadway.

In addition, the effects due to environmental exposure classes were also considered (giving the priority to the beams located at the external sides of the deck).

The objective was to obtain data which could well represent the entire structure and to simplify, at the same time, the operations concerning the laying of connection cables.

The second phase was specifically aimed at the identification of the most sensitive pier. This was made, in a first instance, by applying the simplified procedure described in [22], which is based on the determination of a simplified capacity curve for each pier and the rapid evaluation of the seismic vulnerability in terms of a "capacitive" return period for the relevant Limit State. In order to support the results provided by this procedure, a finite element numerical model of the whole bridge (Fig. 6 – [25]) of the bridge has been also exploited.

The three-dimensional model of the bridge consists of both shell elements (piers and superstructure of the deck) and frame elements (beams cap).

The mechanical properties of materials are those defined in the design phase:

– concrete: characteristic cubic strength $R_{ck} = 55$ MPa; elastic modulus $E_C = 41\,398$ MPa;
– pre-tensioning steel: characteristic tensile strength $f_{\text{ptk}} = 1900$ MPa.

The above mentioned computational model is focused on the piers' response, and cannot be used for the assessment of the pre-stressed beams. In fact, it does not provide an accurate modelling of the deck (and in particular of the beams). Each span, which is simply supported by the adjacent piers, is discretized by shell elements, disregarding the presence of pre-tensioning.

The non-linear static analyses performed on the model have confirmed the results of the simplified procedure, indicating that the critical pier is the one named with the label P-AB in Fig. 7. Considering this result, together with the observations concerning the structural geometry, it was decided to monitor the two spans indicated in Fig. 7 with the letters "A" and "B", and the intermediate pier (label P-AB in the same figure).

In the transversal direction, the monitored beams are the ones indicated with the labels A1, A7, A8, A9, B1, B7, B8, and B9 (six are located on the side more exposed to environmental actions, and two on the other one).

It should be mentioned that the final choice had also the advantage of simplifying the operative installation phase of the system, since the length of the connection cables is quite short.

Each beam has been equipped with 6 SOFO sensors located at relevant positions (end-sections and middle-section, Figs. 8, 9). The sensors were directly fixed on the strands before the concrete casting, and were tested by performing control measurements before and after casting. In all the beams, except for A8 and B8, the 6 SOFO optic sensors have been also coupled with 6 thermocouple.

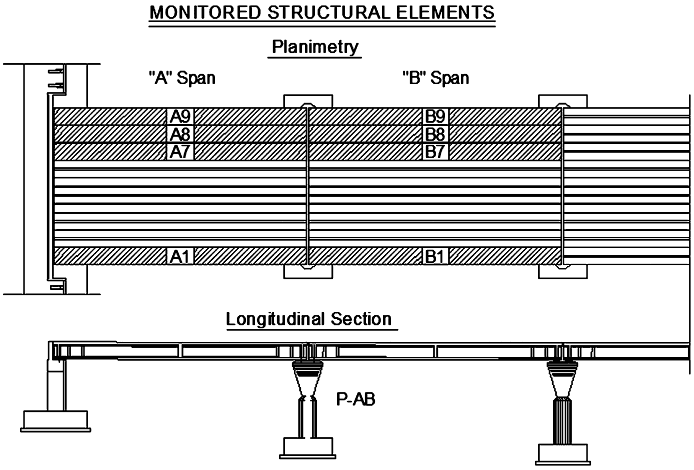

Fig. 7. Identification of the monitored structural elements.

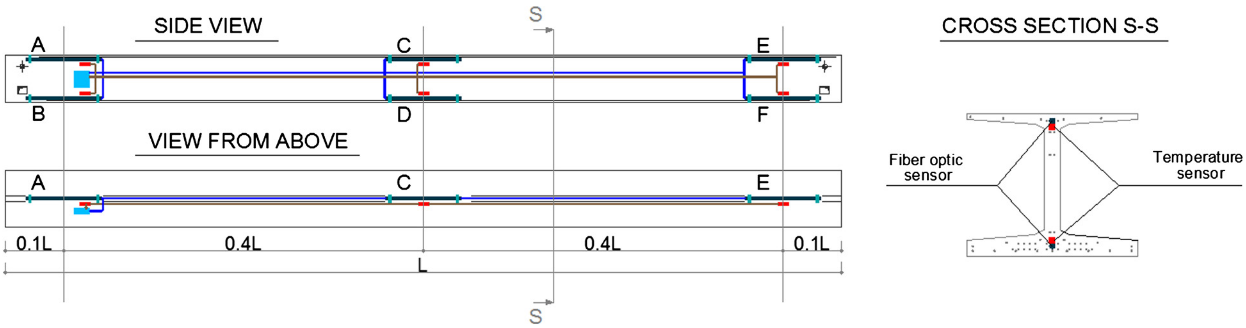

Fig. 8. Position of the sensors on the beam in longitudinal direction and in the cross-section (in the side view, the thickness of the flanges is not indicated for clarity).

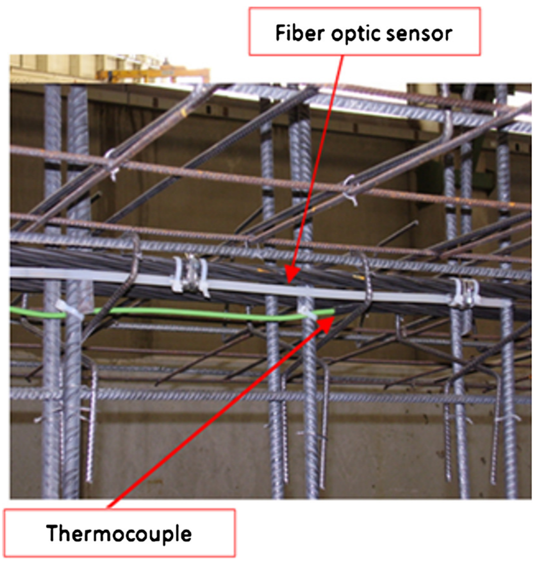

Fig. 9. View of the thermocouple and of the SOFO sensor anchored to the reinforcements before the concrete cast.

The monitoring system includes temperature sensors (thermocouples) aimed at evaluating the effects induced by temperature changes. Bridges are particularly sensitive to the effects caused by temperature changes. In fact, the bituminous layer on the extrados of the deck can reach, during summer, very high temperatures, that significantly influence the defor-

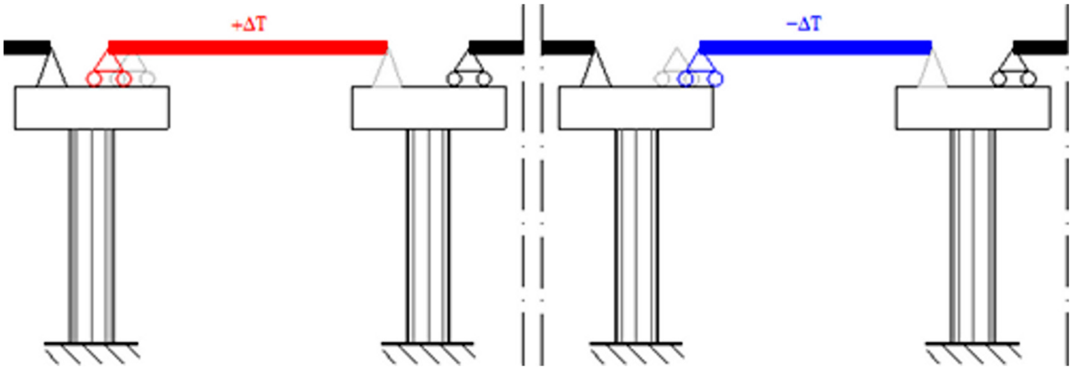

Fig. 10. Simplified static scheme of the bridge deck.

mation field in the supporting structures. Therefore, it was necessary the use of the thermocouples, able to account of the strain rates due to thermal excursion.

In this context, it is known that, in the generic section of the beam, the values of deformation measured by sensors placed on top of the fibers are different from those acquired by the sensors anchored to the lower cables, not only because of the flexural deformation, but especially for the thermal expansion. In order to account for these effects, the single data acquired by the monitoring system should be properly corrected, by removing the strain part related to the temperature variation.

Each acquisition was supplemented by a reading of the temperature, allowing the assessment of the strain variation induced by thermal excursion. This aliquot is then added or subtracted from total one (depending on the sign of the strain – elongation or contraction). The expression adopted for correcting the generic data acquired from single sensor is the following:

$$\varepsilon(t) = \varepsilon_L(t) + \Delta\varepsilon_{\Delta T}(t) \tag{1}$$

where $\varepsilon_L(t)$ is the strain measured by the sensor in the generic reading (L), and $\Delta\varepsilon_{\Delta T} > 0$ if $t - 20\,^\circ\mathrm{C} > 0$; $\Delta\varepsilon_{\Delta T} < 0$ if $t - 20\,^\circ\mathrm{C} < 0$.

The appraisal of the contribution of thermal variation was rather simple, thanks to the peculiar typology of the bridge. In fact, the bridge deck is made by simply supported continuous beams, and the longitudinal section of the bridge can be thence represented by means of the simple static scheme that is hereafter reported in Fig. 10.

4. Acquisition of data

4.1. The monitoring plan

As mentioned in Section 3.2, the implementation of a permanent structural monitoring system allows the achievement of 4 different objectives (OBJ) with regard to the assessment of the structural performance of the bridge in different phases (construction stages; final inspection and acceptance tests on the completed structure; service life of the structure).

The present work is focused on the Objective 2, which is specifically devoted to the control and assessment of the pre-stressed beams during the construction phases, and on the definition of the general methodology for the structural assessment during the service life (which is a part of Objective 4).

After a preliminary verification of the reliability of the installation phase of the sensors, the Objective 2 provides the acquisition of data during the three main construction stages:

1. Launching of the beams.
2. Casting of transoms and concrete slab.
3. Completion of the bridge.

The data recorded by the monitoring system (strain measured on the strands) have been processed in order to identify the actual structural response, and to perform a comparison with the expected theoretical values.

The initial reference time t_0 is assumed to coincide with the launching of the beams.

If "i" and "j" are the sensor anchoring points (anchor pieces of Fig. 1), the generic measurement m_S of the sensor represents a relative displacement between them. During the construction phases, the measured values are:

- $m_S(0)$ = strain measured value at the time t_0 (launching of the beam);
- $m_S(1)$ = strain measured value after the completion of the slab and of the transoms (t_1);
- $m_S(2)$ = strain measured value after the completion of the bridge (t_2).

Table 1
Conventional percentage of the stress losses considered in the model.

Construction stage	Steel relaxation	Concrete	
		shrinkage	creep
"0" – Release of the strands	20%	–	–
"I" – Casting of the concrete slab	70%	30%	30%
"II" – Completion and solidarization of the concrete slab and application of permanent loads	10%	30%	30%
"III" – Final state: all stress losses are fully developed	–	40%	40%

The experimental strain variations at the different times t_1 and t_2 are calculated by the following equations:

$$\Delta\varepsilon_{EXP}(t_1) = \frac{m_S(1) - m_S(0)}{L_s}; \qquad \Delta\varepsilon_{EXP}(t_2) = \frac{m_S(2) - m_S(0)}{L_s} \tag{2}$$

where L_S is the length of the measurement fiber ("active zone").

In the case study, it is $L_S = 300$ mm.

4.2. Loading cases and percentage stress losses

In order to appraise the theoretical strain variation induced by loads and stress losses, the flexural response of the beam in the end-section and in the middle-section has been preliminarily evaluated.

Four different loading cases (indicated with "0", "I", "II", "III") are considered, corresponding to a specific sequence of execution phases and service phases:

- *Phase "0"*: this is the phase immediately after the release of the strands. The resisting section includes only the beam, which must resist to the stress induced by the self weight and by the initial pre-stress;
- *Phase "I"*: this is the phase immediately after the casting of the slab. The resisting section is again the beam alone, and the stress is the same of the phase "0", plus the effect of the slab's weight. A part of the pretension loss, that is already partially occurred, should be considered.
- *Phase "II"*: this is the phase after that the slab is completely integral with the beams and the permanent loads are fully applied. The resisting section is represented by the pre-stressed beam together with the RC slab, and must resist to the stress induced by the whole load, including the non-structural loads. The effect of the pretension should account for a further quota of the final stress loss.
- *Phase "III"*: in this phase, the stress losses in the pre-tensioned strands are fully developed, and the pre-stress value is therefore the final one, including all the stress losses.

In the different phases, a proper quote of the stress loss in the pre-tensioned steel has been considered, in order to account that the pre-tension applied to the strands is not completely transferred to the beam. In the technical literature, the stress losses are usually divided in "*instant*" stress losses and "*delayed*" stress losses, depending on the circumstance that they develop in the same moment of the pre-tensioning of concrete or are gradually developed after a lapse of time. In the present case study, cables are pre-tensioned, and the only possible instant loss is related to elastic deformation.

Additionally, the following delayed phenomena have been considered:

- slackening of the steel;
- shrinkage;
- creep.

The values of the delayed stress losses have been assumed according to the conventional percentages indicated in Table 1, which are referred to the most typical values adopted by the manufacturers.

With regard to this aspect, it is worth remembering that it would be very useful to perform more detailed studies exploiting the resident monitoring system, as outlined in the Objective 1.

5. Data processing

5.1. Numerical comparison

In Fig. 11, the comparisons between the experimental measures and the expected theoretical values – at the different times t_1 and t_2 (as defined in Section 4.1) – are shown for the beams A1, A7, A8 and A9 (the symbols used for the sensors are the same defined in Fig. 9). For brevity sake, the results concerning the monitoring of the other span (B) have not been reported, since they are not significantly different.

The comparison is expressed in an adimensional form, by representing the ratio $\Delta\varepsilon/\Delta\varepsilon_{THEO}^{max}(t_2)$, where $\Delta\varepsilon$ is given by Eq. (2), whereas $\Delta\varepsilon_{THEO}^{max}(t_2)$ is the maximum among all the theoretical strain variation at time t_2. For each of the 4 beams,

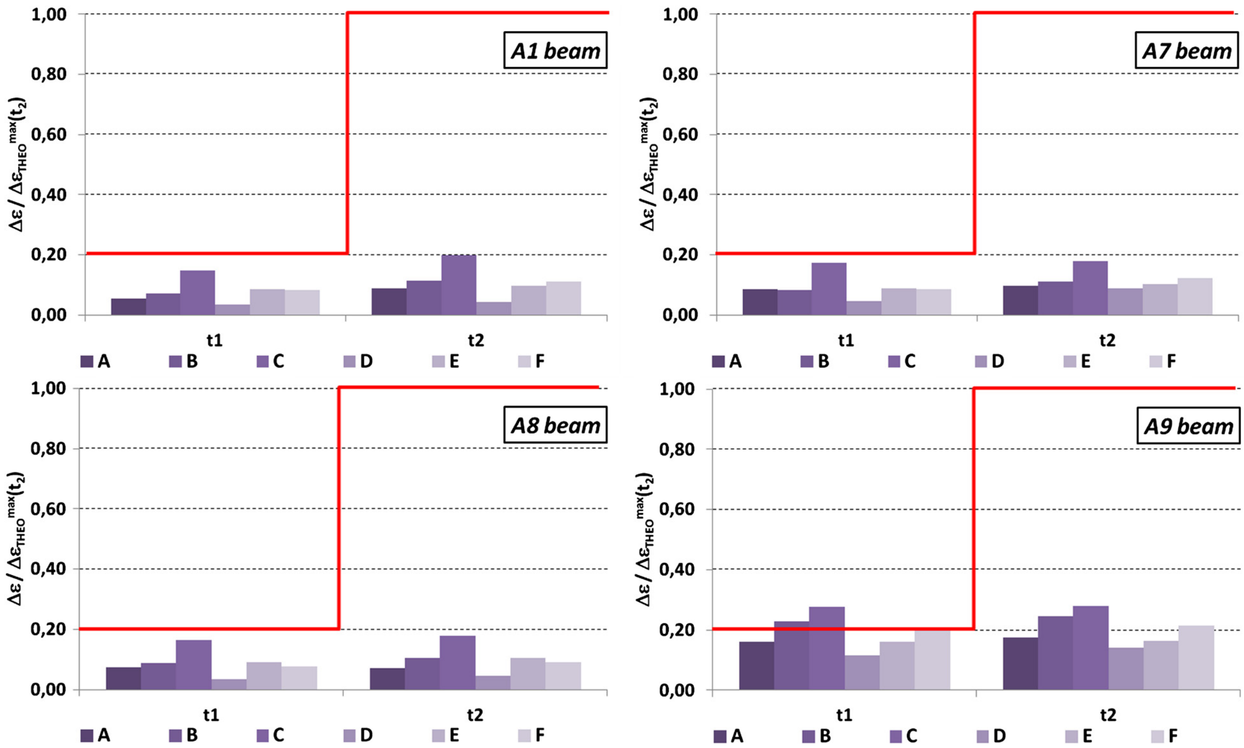

Fig. 11. Comparison between experimental and theoretical values of the strain variation in the different constructive phases.

and for each of the 6 sensors, the blue histograms plotted in Fig. 11 are obtained. Besides, the figure also includes the plot of the "*safety limit*" function (red line), whose points are defined by the ratio $\Delta\varepsilon_{THEO}(t)/\Delta\varepsilon_{THEO}^{max}(t_2)$. This function represents the "theoretical" structural behaviour of the bridge. It is worth observing that the distance between the red curve and the histograms would indicate that the actual deformative behaviour is different from that expected in the project. In the figure, this happens at time t_2 (completion of the bridge) for all the monitored beams. This circumstance is partially related to the purely conventional nature of the stress losses assumed in the calculation of the theoretical stress (see Table 1). At this stage, indeed, it is assumed that the stress variation is equal to 100%, whereas the complete development of the stress losses related to steel relaxation might occur after many years from the realization of the structure. For example, the EC2 (see Section 3.3.2(8)) suggests that the relaxation is fully developed after about 50 years, which is much more than the actual time of the measurement, that coincides with the completion bridge (that is to say, 8 months).

The aim of the *safety limit function* is to point out the positions where the theoretical stress state is exceeded, which could correspond to an abnormal structural response. This happens, in Fig. 11, in the beam A9, for the sensors B and C. Anyway, such a disagreement – which is moreover quite small – should not be considered significant. Firstly, it has to be remembered that the threshold provided by the safety limit function is based on conventional assumptions about the stress loss rate during the monitored period. Moreover, the moderate difference encountered between theoretical and experimental values at time t_1 is compatible with the possible variation range in the choices made in Table 1 and should not induce a real concern about the bridge safety.

5.2. Some remarks about the results

Available data about the monitoring, by now, concern the construction phases, up to the completion of the bridge. The data recorded during the final test of the bridge are still not available, whereas the monitoring of the service conditions is still in progress. An in depth investigation of these aspects (which involve the Objectives 3 and 4) will be provided in a forthcoming research work.

It is possible to provide some interesting indications about the expected structural behaviour at the time t_3 (that is actually very far from the completion of the construction), as shown in Fig. 12. This diagram actually represent a qualitative, general procedure for the systematic assessment of the structural behaviour during the service life by means of a real-time monitoring system. The actual diagram of the structure shall be specified on the basis of the data provided by the resident monitoring system for the specific element, during its service (the diagram is progressively updated during the service life of the structure). The solid line qualitatively represents the theoretical trend of the strain variation, under the hypothesis that the degradation of the materials is negligible (such an hypothesis could be unrealistic, but the results are anyway significant in view of the definition of the general procedure of assessment). The slope of the line which represents the theoretical

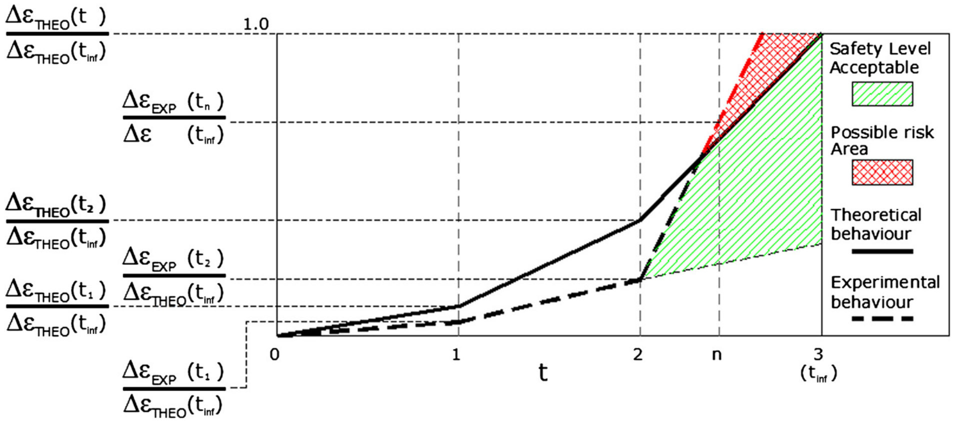

Fig. 12. The graphical interpretation of the results.

behaviour grows as long as additional strain variations occur (for example, because the constructive stage provides the application of a new load).

The dashed line represents the "experimental" structural behaviour, and is built according to the strain variations measured by the sensors.

At this point, the comparison between the two curves provides an indication about the structural integrity for the specific monitored element. If the data recorded by the monitoring system define an experimental curve which is above the theoretical one (red area), a deviation from the expected structural response is occurring, indicating a possible abnormal phenomenon. After all, the systematic comparison allows the prompt detection of abnormal structural behaviours, allowing to carry out the necessary additional inspections and verifications, planning the possible necessary actions and, at the same time, keeping under control the evolution of the phenomenon.

6. Conclusions

The paper shows that the use of optic fiber sensors allows the implementation of effective systems for the real-time structural monitoring, particularly useful for the control of strategic buildings and infrastructures. The technique has been implemented in a bridge recently built in the city of Bari, Italy, by installing the sensors over the pre-tensioned strands within the precast beams, and over the mild steel within the piers, allowing the control of the execution phases and the monitoring of the actual conditions of the materials. The monitoring is still in progress, and the measurements acquired, until now, have allowed to control the execution phases and the actual conditions of the materials. Data were acquired during the significant phases of the constructive process (indicated by the labels t_0, t_1 and t_2). The comparison between the recorded values and the theoretical prediction has confirmed the robustness of the system, providing a tool for controlling the effects of deformation over time (see Fig. 11).

The numerical comparison (expressed by the ratio $\Delta\varepsilon_{THEO}(t)/\Delta\varepsilon_{THEO}^{max}(t_2)$) has shown that after the completion of the bridge (ideal time t_3), the delayed time effects were only partially developed, as it could be expected because of the very short time elapsed between the measurements and conclusion of the works. An extension of the graphical comparison has been also proposed to the service life of the structure (Fig. 12), in order to outline the future application of the methodology in the regular maintenance programs of the bridge. If during the periodical monitoring the strain variation (expressed in the figure in the adimensional form) exceeds the safety threshold (i.e., the "safety limit function" defined in Section 5.1), this can indicate an anomaly in the structural behaviour caused by external phenomena which were not predicted in design (for instance, degradation of the materials or exceptional actions). It is thence possible to take prompt actions in order to further investigate the problem and, eventually, implement the necessary interventions. The simple graphic procedure described in Section 5.1 is proposed as an operational methodology to be applied within the maintenance programs of the structures.

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