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Construction assessment and long-term prediction of prestressed concrete bridges based on monitoring data

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Abstract

The long-term assessment of large civil infrastructures such as prestressed concrete bridges is a challenging task. The real time history related to the phased construction, the influence of the adopted construction method, the characterization of the employed concrete and the environmental conditions are crucial aspects for accurate predictions based on Finite Element Models. Data from long-term monitoring systems have been used to improve the quality of those predictions, namely to validate the design assumptions, to calibrate the structural model, and to update the safety coefficients.

The new bridge over the Tagus River in Portugal, the Lezíria Bridge, comprehends a 1,700 m long viaduct, which was erected using a movable scaffolding system. A long-term monitoring system was implemented and measurements were taken since the start of the construction. Based on these measurements, this work presents the analysis strategy adopted to compute the long-term behaviour based on a Finite Element Model. A detailed scanning of all information with relevance for the viaduct behaviour was performed. A full discussion concerning the real long-term behaviour is made, focusing the differences between the measurements and the results obtained with the numerical model, namely the trends due to shrinkage and creep and the variations due to the temperature. Finally, concerning to the viaduct lifetime, the predicted values for the measured parameters are updated, since the predictions based on the European standard presents significant bias to be used as reference values regarding the viaduct surveillance.

Keywords: Monitoring, concrete bridges, movable scaffolding system, shrinkage, creep, relaxation, temperature, prediction, structure lifetime, maintenance.

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1. Introduction

The long-term assessment of large civil infrastructures such as prestressed concrete bridges is a challenging task. One of the most common and reliable strategies is the use of Finite Element Models (FEM). Although, nowadays, the short-term behaviour, e.g. load tests, can be interpreted with FEM analysis with satisfactory accuracy, the long-term prediction is not so straightforward. The updated bridge designs, the continuous change of the materials properties and the advances in the construction methods employed in the civil engineering domain are aspects with direct influence in the long-term response. Therefore, and in addition to the geometric and materials properties, all other aspects with significance in the bridge behaviour must be taken into account in the analysis strategies to get more reliable predictions and to reduce deviations. The real time history related to the phased construction, the influence of the adopted construction method, the accurate characterization of the employed concrete and the environmental conditions are some of the most relevant aspects.

As regard the construction methods, one of the most commonly used in the construction of cast in situ concrete bridges is the movable scaffolding systems. A travelling steel structure, which supports the formwork that gives shape to the bridge, is the basis of these systems. It is frequently applied to multi-span bridges over difficult terrain or water, where the launching girder moves forward span-by-span, supported on the piers. The application of this system is profitable for long bridges with spans ranging between 30 to 40 m and a high degree of repetition. The movable scaffolding is supported on the bridge piers and therefore, an interaction between the girder and the bridge exists, which can lead to higher deformation and stress levels than the ones that will occur during the service life. This is why the bridge behaviour during this transitional period must be controlled. Measurements obtained since early ages with appropriate sensors would be desirable to get a more comprehensive knowledge of the real behaviour of bridges constructed with these systems. However, databases with this type of information are scarce.

As far as the concrete properties are concerned, several studies show that long-term predictions differ significantly from the observed response, mainly due to the evaluation of shrinkage and creep during the design phase [1-3].

Besides the rheological effects of concrete, the variability concerning the environmental conditions, namely the humidity and temperature, are also critical for an accurate evaluation of the effective long-term response [4-6].

Data from long-term monitoring systems have been supporting the aforementioned studies, which demonstrate the importance of these systems to improve the knowledge concerning the effective structural behaviour. Initially, these measurements were provided to detect in advance any possible risk factor during the construction of the bridge, and thus prevent any resulting accident [7]. With the technology evolution, the monitoring systems have become more and more powerful, offering information on structural integrity, durability and reliability, and ensuring optimal maintenance planning and safe bridge operations [8]. In fact, monitoring data have been contributing to validate the design assumptions, to calibrate the structural model, and to update the safety coefficients.

The recent construction of a major bridge in Portugal, the Lezíria Bridge, comprises a 1,700 m viaduct erected using a movable scaffolding system, which offered the opportunity to observe its long-term behaviour and compare with that obtained with a numerical model. The measurements were performed by a permanent monitoring system installed in the bridge since the beginning of the construction, devoted to the bridge surveillance and maintenance [9].

This work presents a numerical model implemented to the prediction of the long-term response of the viaduct, which was calibrated with measurements obtained during the construction phase and the load test. The effective mechanical and time dependent properties of concrete and prestressing steel were considered, as well as the effective prestress forces and the real construction sequence. A full discussion concerning the real long-term behaviour is made, focusing the differences between the measurements and the results obtained with the numerical model, namely the trends due to shrinkage and creep and the variations due to the temperature. Finally, concerning the viaduct lifetime, the predicted values for the measured parameters are reviewed, since the predictions based on the European standard presents significant bias to be used as reference values regarding the viaduct surveillance.

2. Case study

2.1. Description of the structure

The 11,670 m total length of the Lezíria Bridge is composed of three structures: (i) the north approach viaduct 1,700 m long; (ii) the main bridge structure, crossing the River Tagus, with a total length of 970 m; (iii) and the south approach viaduct, with a total length of 9,160 m (Fig. 1). From the three substructures, the north approach consist of three elementary viaducts, V1N, V2N and V3N, with spans of 33 m, except at the railway crossing in which the largest span is 65 m long and is partially formed by a box girder. The deck has a total width of 29.95 m, where 2×1.15 m is sidewalk technical paths. As regards the V2N viaduct, the expansion joints are spaced at 470 m to limit the deck expansion and control the maximum deformations imposed to the vertical elements [10].

The viaduct deck is generally supported by four alignments of circular piers-pile with 1.5 m of diameter and lengths that can reach 40 m deep (Fig. 2-a). The construction process adopted to construct the viaducts was a self-supporting movable scaffolding supported on the piers. This movable structure was recurrently mounted and dismantled. After positioning, works related with reinforcement arrangement, metallic sheaths and anchorage systems installation were performed. Then, the span construction was proceeded with the concrete pouring. Finally, the prestressing cables were tensioned and the movable scaffolding was moved to the next span (Fig. 2-b and c). The viaduct construction was finished with the concreting of the service sidewalks, the bituminous layer, positioning of expansion joints, exterior safety barriers, railings and border beams. Fig. 3 presents the elevation of the viaduct under analysis, V2N, while Table 1 presents the time history of its construction.

2.2. The monitoring system

The Lezíria Bridge is equipped with an integrated monitoring system to support the management and surveillance of the structure. In what it concerns to the V2N viaduct, two zones were instrumented: (i) Zone 1, referring to the first three spans between piers TP1 and P3 and (ii) Zone 2, referring to the last three spans between piers P10 and TP2

(Fig. 3). Selected cross sections in these two zones were instrumented with suitable sensors.

Focusing on Zone 1, and in accordance with Fig. 4, the sensors installed were vibrating wire strain gauges in three deck sections (P1P2, P2 and P2P3) to measure the concrete deformations (CD). Fig. 5 depicts the typical instrumentation of girder sections, each section with four vibrating wire strain gauges, two in each alignment A and B. The strain gauges have an internal thermistor, which was used to measure the concrete temperature (CT). The strain values captured by the vibrating wire strain gauges were corrected by eliminating the effect of the free thermal deformation of the wire and the concrete. For this purpose, the thermal dilation coefficient took the value $11 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ in the case of the wire (given by the manufacturer) and $7.9 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ in the case of the employed concrete [11]. Sensors measurements were generally taken since the concrete pouring, and when this was not possible, reference readings were taken before the concreting.

The bearing displacements (BD) at the viaduct extremities were also monitored (see Fig. 4), but only after the end of construction. The vertical displacements (VD) of sections P1P2 and P2P3 were measured during the load test, with reference to the soil as presented in Fig. 4. Two load cases, LC1 and LC2, are also indicated in Fig. 4, which are used later in the results discussion. These load cases, with four trucks placed at the girder cross section, correspond to the maximum deflection of the spans P1P2 and P2P3 respectively.

Regarding the long-term behaviour, strains and temperatures were also recorded in six prisms with dimensions of $15 \text{ cm} \times 15 \text{ cm} \times 55 \text{ cm}$, with two long faces unsealed. Four of these six prisms were used for shrinkage measurements while the other two prisms were used for creep measurements. The prisms were cast with the same concrete of the girder and kept under to the same curing conditions. In order to get a more comprehensive characterization of the concrete employed in the deck girder, two shrinkage prisms were cast with concrete of Zone 2, while the remaining prisms (shrinkage and creep) were cast with concrete of Zone 1 (see Fig. 3). A complete description of the monitoring and data acquisition system installed in the Lezíria Bridge can be found elsewhere [9].

3. Finite Element Analysis

3.1. General considerations

A non-linear finite element analysis was performed by using the general-purpose finite-element code DIANA [12]. Structural discretization was carried out using beam finite elements of three nodes, approximately 1m long, based on an isoparametric formulation [13]. Only one alignment of girders, alignment A (see Fig. 2), is modelled in the two dimensional analysis, as the loading and slab width over each alignment is similar. In this model, both geometry and loading are symmetric to the girder axis. The reinforcement, both ordinary and prestressed, was modelled using embedded reinforcement elements. In this way, prestress losses due to concrete creep and shrinkage as well as relaxation of the prestressing cables are automatically accounted for. The piles-soil interaction was also modelled with elastic springs. A phased analysis with 56 stages was performed, where the model was changed by including new elements and/or modifying the connection to the supports at each new stage. The sequence adopted in the analysis follows the real chronology observed during the construction (Table 1) [14]. Errors are inevitable in the construction of numerical models concerning large structures, namely due to data input [15]. Therefore, in order to mitigate potential errors, CAD tools were specifically developed and used during the scanning of the drawing pieces, with benefits on time expended [16].

3.2. Structural modelling

The construction of the geometric model was based on the final drawings of the bridge project [10]. The structural elements, piles, piers and viaduct girder, were reduced to their axis as presented in Fig. 6-a.

The ordinary reinforcement of all structural elements was taken into account with layers of reinforcement near the concrete faces. An additional reinforcement layer, matching the girder axis, was modelled to take into account the effect of the web reinforcement in the concrete delayed deformations. The embedded prestressed cables in the girder concrete were precisely modelled with parabolic elements, in order to have a rigorous evaluation of the prestressing losses (Fig. 6-b). To facilitate the analysis of the viaduct behaviour since early ages, the movable scaffolding system was also modelled with beam elements supported in the viaduct piers. Globally, the numerical model has 992 beam elements, 396 linear reinforcement elements (ordinary

reinforcements), 60 parabolic reinforcement elements (prestress cables), 561 spring and 14 supports.

3.3. Concrete modelling

In concrete structures, to obtain a correct prediction of the structural behaviour, an accurate modelling of the material properties is required, namely the concrete. Therefore, the definition of the mechanical properties of concrete were based on the tests of 150 mm cubes, which were performed during the construction [14], and in measurements taken from the reference concrete prisms to evaluate the shrinkage and creep.

The expressions proposed by the European code Eurocode 2, EC2, were considered to describe the time variation of the concrete properties, namely the modulus of elasticity, creep and shrinkage [17]. Whenever possible, the value of the parameters involved in those expressions was obtained from the measurements. Moreover, concretes with different mechanical properties were considered in the FEM analysis to take into account the phased concrete pouring performed during the viaduct construction.

3.3.1. Evolution of concrete compressive strength

The consideration of the concrete compressive strength evolution is mandatory for a long-term analysis, since it is correlated with the evolution of the concrete modulus of elasticity. The concrete compressive strength was calculated based on compressive tests performed in 15 cm cubes. An extensive study was performed to characterize the compressive strength of different structural elements, namely piles, piers and girder. Since the cylinder strength was not experimentally evaluated, it was taken as 82 % of the observed cube strength as recommended by EC2 [17]. Table 2 summarizes the statistical information concerning the compressive strength at 28 days.

Based on these results, the compressive strength at a given age, $f_{cm}(t)$, is given in the EC2 by Eq. (1).

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad , \quad \beta_{cc}(t) = \exp \left[s \cdot \left(1 - \sqrt{\frac{28}{t}} \right) \right] \quad (1)$$

where t represents the concrete age in days, s is a cement-hardening coefficient that characterizes the evolution of the concrete strength, and f_{cm} is the mean value of the concrete compressive strength, at the age of 28 days. The parameter s was determined by a curve fitting procedure, which consisted in minimizing the mean square error between the cube test results (at different ages) and the equation (1). Fig. 7 illustrates the curve fitting results for the concrete in the instrumented zone, namely of pile 2, pier 2 and deck span P2P3 (Fig. 3), whilst Table 3 summarizes the values obtained for those parameters, grouped by different structural elements.

3.3.2. Modulus of elasticity

The determination of the tangent modulus of elasticity, E_c , was based on the observed concrete compressive strength, by means of the Eq. (2).

$$E_c = 1.05 \cdot 22000 \cdot (f_{cm,cyl}/10)^{0.3} \quad (E_c \text{ and } f_{cm,cyl} \text{ in MPa}) \quad (2)$$

where $f_{cm,cyl}$ represents the mean value of the concrete cylinder strength at the age of 28 days (Table 2). According with EC2, the evolution of the modulus of elasticity correlates to the time variation of the compressive strength (determined above), and is given by Eq. (3). Table 3 summarizes the average values, depicted by structural element type.

$$E_c(t) = \beta_E(t) \cdot E_{c(28)} \quad , \quad \beta_E(t) = \exp \left[\frac{s}{2} \cdot \left(1 - \sqrt{\frac{28}{t}} \right) \right]^{0.5} \quad (3)$$

3.3.3. Shrinkage

The total shrinkage strain, ϵ_{cs} , is set by two parts as expressed in Eq. (4): the drying shrinkage strain, ϵ_{cd} , and the autogenous shrinkage strain, ϵ_{ca} . The drying shrinkage strain develops slowly, since it is a function of the migration of the water through the hardened concrete, while the autogenous shrinkage strain develops during concrete hardening phase [17]. The mathematical models given by EC2 for the drying and autogenous shrinkage are presented in Eq. (5) and Eq. (6), respectively, and they are expressed through a multiplicative model with a nominal coefficient, $\epsilon_{c,\infty}$ and a time factor, $\beta_s(t)$.

$$\varepsilon_{cd}(t) = \varepsilon_{cd}(t) + \varepsilon_{ca}(t) \quad (4)$$

$$\varepsilon_{cd}(t) = k_{cs,0} \cdot \varepsilon_{cd,\infty} \cdot \beta_{ds}(t, t_s)^{k_{cs,t}} \quad (5)$$

$$\varepsilon_{ca}(t) = \varepsilon_{ca,\infty} \cdot \beta_{as}(t), \quad (6)$$

where t is the time (in days) since drying begins, t_s . Additionally, $k_{cs,0}$ and $k_{cs,t}$ parameters were introduced in Eq. (5) so that the drying shrinkage model could be scaled and adjusted to the experimental results obtained from the concrete prisms [18]. In fact, the autogenous shrinkage was not considered in the fitting problem, because the major part develops in the early days after casting [17]. However, the autogenous shrinkage was not neglected because the measurements collected from the concrete prisms had started since the concreting. Instead, its effect was removed from the measurements by subtracting a quantity expressed by Eq. (6) to allow the fitting process to the drying shrinkage model, and afterwards it was added. Therefore, the model presented in Eq. (4) was fitted to the strain measurements performed in the concrete prisms. Additionally, an average temperature of 16 °C and a relative humidity of 65 % were considered, based on the collected measurements.

Fig. 8 presents the results achieved for the shrinkage prisms of Zone 1 and Zone 2 (see Fig. 3). A more realistic model is obtained with the fitted EC2 model, being clear that the EC2 model overestimates the shrinkage deformations. The shrinkage deformations are almost equal for both zones, around the 300 $\mu\epsilon$ after almost 2000 days ($k_{cs,0} \cong 0.53$), while the evolution in the early ages are different ($k_{cs,t} = 0.50$ for prisms of Zone 1 and $k_{cs,t} = 1.73$ for the prisms of Zone 2). Moreover, the results respecting the Zone 2 are better adjusted for long-term while the results concerning the Zone 1 exhibit a better adjustment in the first ages. This seems to be explained by the scarce measurements obtained for the prisms of the Zone 2 during the first 250 days. In fact, if the shrinkage measurements during this period for Zone 1 are disregard, the obtained fitted model is very similar to the one obtained for Zone 2 ($k_{cs,t} = 0.56$ and $k_{cs,t} = 1.85$). In this context, different models are necessary for a correct interpretation of both the construction stage and the long-term behaviour.

The cross-sectional shape influences the evolution of the shrinkage strains and this dependence is commonly expressed by the notional size [17]. The notional size of the structural elements of the viaduct ranges between 600 mm (Zone 1) to 1070 mm (Zone 2), which are significantly higher than the shrinkage prisms that are 150 mm. The

strategy employed in this work was to derive the shrinkage curve of the structural elements based on their effective notional size and the fitted curve obtained for the shrinkage prisms (Fig. 8). In addition, the effect of the bituminous layer was taken into account in the notional size calculations due to the waterproofing effect on the upper face of the girder. Although this might not be entirely true during the construction stage, this is an essential issue to predict the structure behaviour for viaduct lifetime. Finally, in order to take into account the beam thickness variation (see Fig. 5), three different shrinkage curves were calculated with different notational size for each of the 13 spans and therefore, 3×13 shrinkage curves were calculated to be considered in the input file as presented in Fig. 9.

3.3.4. Creep

According with the EC2, the creep deformations of concrete, $\varepsilon_{cc}(t, t_0)$ at a generic time t for a constant compressive applied stress at the age t_0 is given by Eq. (7), where $\varphi(t, t_0)$ is the creep coefficient and $\varepsilon_c(t_0)$ is the instantaneous deformation due to the applied compressive stress. Moreover, EC2 sets the creep coefficient according to Eq. (8).

$$\varepsilon_{cc}(t, t_0) = \varphi(t, t_0) \cdot \varepsilon_c(t_0) \quad (7)$$

$$\varphi(t, t_0) = k_{cc,0} \cdot \varphi_0 \cdot [\beta_c(t, t_0)]^{k_{cc,t}} \quad (8)$$

where φ_0 is the notional creep coefficient and $\beta_c(t, t_0)$ is a coefficient to describe the development of creep with time after loading. Similar to the shrinkage, the additional parameters, $k_{cc,0}$ and $k_{cc,t}$ were introduced in Eq. (8) so that the creep model could be scaled and adjusted to the experimental results [18, 19]. Therefore, the EC2 model was fitted to the strain measurements performed in the concrete prisms. Again, a more realistic model is obtained with the fitted model as it is presented in Fig. 10, where it is clear that the EC2 model overestimate the creep deformations. In fact, differences between observed values and code predictions were expected since the employed concretes contain superplasticizers and pozzolanic materials, whose effect in the composition cannot be accurately reproduced by code formulae [20]. Similar to the strategy adopted for the shrinkage, the notional size was considered to scale the creep function to the structural elements of the viaduct.

3.4. Prestress cables modelling

An exhaustive scanning was carried out to characterize the modulus of elasticity of the employed prestress steel, based on the manufacture specifications [14]. This information was considered into the numerical model by considering the average value of the prestress cables used for each span. Globally, an average value of 199.7 GPa was achieved, which satisfies the EC2 limits of 195 GPa to 210 GPa [17]. Moreover, the minimum and maximum values obtained were 193.0 GPa and 209.0 GPa respectively, with a coefficient of variation of 1.7%. Taking the fact that the prestressed cables have low relaxation and were conducted in flexible metal ducts, it was adopted the relaxation class 2 [21]. Based on the recommendations given by Model Code 2010 [22], the wobble coefficient K, per meter, was fixed to 0.05 and the coefficient of friction, μ , was set to 0.19.

3.5. Soil modelling

The interaction between the piles and the soil was modelled with elastic springs based on the Winkler model. The spring stiffness is proportional to the influence area of the spring, A_{inf} , and the subgrade reaction module, k_s [23]. The A_{inf} is defined by the pile diameter, ϕ , multiplied by an influence length, L as expressed in Eq. (9). As regards the k_s value, it was evaluated based on field tests performed by the constructor [24]. Table 4 summarizes the average values with interest for this work, which were used to compute the stiffness of the soil springs. The depth values are reference values, which in reality they are variable along the viaduct.

$$k_v = k_s \cdot A_{inf} \quad , \quad A_{inf} = \phi \cdot L \quad (9)$$

3.6. Loading

The loads used in the analysis were: (i) self-weight of the reinforced concrete, with a value of 25 kN/m³; (ii) effective forces applied in the prestressing cables based on elongation measurements, which were estimated to be, in average, 4 % lower than the design values and corresponding to a stress level of approximately $0.71 \cdot f_{puk}$; (iii) superimposed dead loads due to the bituminous layer, border beams, service sidewalks and safety barriers with an estimated value of 28.3 kN/m; (iv) trucks fully loaded used

in the load test with an average weight of 32.1 ton (314.6 kN) each and a standard deviation of 0.4 ton (4.2 kN).

4. Results and discussion

4.1. Load test

Any prediction obtained by a numerical model presents deviations to the real behaviour. Even tough, it is always important to have a numerical model capable of representing, as much as possible, the effective behaviour of the structure in order to get reliable predictions. The best strategy to achieve this objective is validating the numerical model with measurements obtained during load tests, taking profit that the loading is perfectly known and controlled a priori. In this context, the numerical model was calibrated, and some parameters with direct influence in the viaduct behaviour were evaluated namely: (i) soil stiffness, (ii) piles with and without metallic casing, (iii) axial deformability of piles and (iv) dimensions of the pier-deck intersection zone.

Based on the calibrated model, Fig. 11 presents the results concerning the load test (see Fig. 4) for the curvatures (CUR) and the vertical displacements (VD) at sections P1P2 and P2P3. For the maximum values, the vertical displacements results show a better agreement than the ones obtained for the curvatures. This can be explained by the different parameters that are being analysed. The curvature, which is calculated based on the strain measurements, CD-1I and CD-1S (see Fig. 4), provide local information at the viaduct cross section. Moreover, it is highly influenced by the properties of the concrete that embeds the strain gauges. On the other hand, the vertical displacement reflects a global response being less susceptible to local properties. Even though, for the load configurations that led to maximum displacements for sections P1P2 and P2P3, with respect to the load cases LC1 and LC2 (see Fig. 4), the maximum errors were 7 % and 4% respectively. To obtain these results, the calibrated model considers: (i) average values for subgrade reaction module, k_s (Table 4), (ii) piles with metallic casing, (iii) partial deformability of piles and (iv) effective dimensions of the pier-deck intersection zone. In fact, the non-consideration of the effective dimensions of the intersection zone of pier-deck would cause an error increase of almost 8 %, while the non-consideration of the metallic casing would result an error increase of 2 %. The axial deformability of the piles has an influence of 3%. Therefore, if the aforementioned errors are combined,

the errors could increase from the obtained 4 % and 7 % to almost 17 % and 20 % respectively, which is not acceptable for reliable predictions to support the management and surveillance of the viaduct.

4.2. Construction of the viaduct deck

As aforementioned, the girder construction can be described by the following phases: (i) positioning of the movable scaffolding – phase 0, (ii) reinforcement placement and concrete pouring – phase 1, (iii) tensioning of the prestressing cables – phase 2, (iv) release of the movable scaffolding and moving forward – phase 3. Fig. 12 shows the measurements obtained in section P2P3 (see Fig. 4), where the different phases of the construction process can be clearly identified. The most relevant change occurs at 07/08/2006 when the prestressing cables of span P2P3 were tensioned (phase 2), which caused a variation of approximately, $\varepsilon_c = -300 \mu\epsilon$, in the strain gauge CD-II (see Fig. 5). Short time later, a compression decrease was observed in this strain gauge, caused by the release of the movable scaffolding (phase 3). This last variation denotes that the span weight was partly supported by the movable scaffolding even after the prestressing. The measurement of the strain gauge also shows minor offsets at 11/08/2006 and 13/08/2006, corresponding to the construction phases 1 and 2 of the next span P3P4. As far as the evolution of the concrete temperature (CT) is concerned, different patterns are observed for the two thermistor of the vibrating wire sensors CD-II and CD-IS (Fig. 4). Although both sensors are approximately at the same distance of the concrete surface, the sensor at the top, CT-1S, is more prone to the daily thermal variations. The movable scaffolding framework can explain the initial differences between both sensors. The framework bounds all faces of the girder excepting the top face, which facilitates the heat dissipation. Even though, after the end of phase 2 the daily temperature variation is greater at the top face due to the sun exposure, while the bottom face that is always on shade. The time-dependent effects of concrete are also visible by smooth variations of the strain measurements along time.

Considering the shrinkage curves obtained based on the concrete prisms of Zone 1 (Fig. 9-a) and the fitted model for creep deformations (Fig. 10), the results obtained from the numerical model are also plotted in Fig. 12-a. A good agreement with the strain measurements is achieved. The zero-reference for the strain measurements is difficult to establish however, for the present study, it was considered two days after the

concreting. At this stage, the concrete temperature is decreasing yet, and approximately, at half path of the temperature stabilization, where it is expected a full hardening behaviour of the concrete. The most unsatisfactory agreement is observed for CD-1I for the period between the phase 2 of the span P2P3 and the phase 1 of the span P3P4, which can be explained by the temperature variation at the bottom of the deck girder. The temperature is still decreasing due to the heat dissipation of concrete hydration. Therefore, non-linear profiles of the temperature through the section height exist and additional mechanical deformations are originated. The results herein presented do not take into account the concrete hydration effect, being the possible cause for the less satisfactory agreement during this period.

In correspondence to Fig. 12-a, Fig. 12-b presents the stress variation in section P2P3 given by the numerical analysis. The minimum stress occurred at the bottom face with a value of $\sigma_c = -8.9$ MPa, respecting to phase 2 of the construction of the span P2P3. In fact, as aforementioned regarding the strain results, the deformability of the movable scaffolding is partly responsible for this result. After the release of the movable scaffolding the stress decreases to, $\sigma_c = -7.2$ MPa and therefore, it can be concluded that the deformability of the movable scaffolding leads to a stress increase of approximately 24%. Recent works show that it is possible to eliminate this undesirable stress increment by using organic prestressing in the scaffolding systems, i.e. a control system that fosters the design of lighter structures and simultaneously enables enormous service deflection reductions [25]. Even though, along the construction process, the entire section was subjected to compressive stresses that were always inferior to $0.45 \cdot f_{ck}(t)$ and therefore, the concrete service stress limit was not exceeded [17].

4.3. Long-term behaviour

As regards the long-term behaviour, Fig. 13 presents the results concerning the observation of sections TP1, P1P2, P2 and P2P3 (Fig. 4), compared with the corresponding numerical results. To improve the results discussion, three different time dependent approaches are used for concrete: (i) based on the EC2 models; (ii) based on the EC2 models fitted for the construction analysis (Fig. 9. a)), which were the basis of the results presented in Fig. 12, and; iii) the EC2 models fitted for the long-term analysis (Fig. 9. b)). Taking the measurements as baseline, the overestimation committed by the EC2 models is confirmed, while the results obtained with fitted

models lead to a significant improvement in the results quality. This is clearly evident in Fig. 13-a, where the trend, disregarding the variation due to the temperature effect, of the measurement on the bearing displacement at section TP1 (see Fig. 4) presents a better agreement with data obtained using the fitted models than with the EC2 models. Similar results were obtained for the bearing displacement at TP2 section (see Fig. 4). This result has a particular importance for the model validation, because it reflects a global behaviour of the viaduct, while the strain results are influenced by the local properties and conditions at corresponding cross sections. As regard the strain results, although they exhibit a good agreement, some deviations must be explained. The sensors placed in the concrete near the top face of the deck girder, CD-1S, of the mid-span sections P1P2 and P2P3 (Fig. 13-b and Fig. 13-d) are highly influenced by the solar radiation and the rainfall and consequently, they are subjected to particular conditions of temperature and humidity. Observing carefully the pattern evolution of the strains measurements in Fig. 13-b and Fig. 13-d, a significant increase of the concrete shrinkage after the construction end is observed as well as a decrease of the daily variation. In fact, the deck bituminous layer placed at the end of the construction can explain this change of pattern. The bituminous layer prevents the humidity to reach the top face of the deck girder as well as lead to an increase of the concrete temperature due to the heat absorption. Therefore, with less humidity reaching the concrete and higher range of temperatures, an increase of the shrinkage deformations is expected. Another aspect that can explain the differences is the notational size of the deck girder. The higher the notational size, slower is the shrinkage evolution. In fact, the notational size of the structural elements of the viaduct for Zone 1 ranges between 600 mm to 900 mm, which is significantly higher than the shrinkage prisms that are 150 mm. The shrinkage of the structural elements was in fact extrapolated based on the measurements of the concrete prisms (Fig. 9) and therefore, errors associated with this procedure must be taken into account in the discussion of results. Similar conclusions were obtained by other authors studying different instrumented bridges in Portugal [2, 18].

Comparing the two results obtained based on the fitted models presented in Fig. 9. a) and Fig. 9. b), they are complementary. As expected, a better approach during the construction phase is obtained based on the shrinkage results presented in Fig. 9-a, while the long-term response is better interpreted with the shrinkage deformations presented in Fig. 9-b. These results are in conformity the fitting curves presented in Fig. 8.

As regards the influence of the relaxation of the prestress steel, its effect is low. For the structure lifetime, 100 years, its contribution is lower than 2.0%.

4.4. Prediction for the viaduct lifetime

In spite of the long-term behaviour is mostly conditioned by the time-dependent properties of concrete, namely shrinkage and creep, the environment conditions has also an important contribute, namely the temperature variation, as it can be noticed by the seasonal variation of the measurements presented in Fig. 13. Besides the EC2 models for shrinkage and creep overestimate the long-term response of the viaduct, laboratory tests revealed that the thermal dilation coefficient of the employed concrete is approximately $7.9 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ [11], as aforementioned, which is 21 % lower than the $10 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ specified in Eurocode 1 - EC1 [26]. This value is consistent with results obtained by other authors [2, 27]. As a result, the predictions based on the European standard rules are unrealistic to be considered as the expected values for the sensors measurements and they should be calibrated focusing the viaduct lifetime.

In this context, the predictions with best performance for long-term purposes previously presented in Fig. 13, i.e. EC2 model fitted for long-term analysis (Fig. 9. b)), were extended for the viaduct lifetime of 100 years. Additionally, the temperature effect was taken into account according with the specifications for thermal actions on bridges presented in the EC1 and the national annex for the Portuguese case [26]. Thus, the characteristic values adopted for the minimum and maximum uniform temperature components were $T_{\min} = 0^{\circ}\text{C}$ and $T_{\max} = 40^{\circ}\text{C}$, respectively, while for the differential temperature components the values considered were $\Delta T_{\text{M,cool}} = -5^{\circ}\text{C}$ and $\Delta T_{\text{M,heat}} = +15^{\circ}\text{C}$ respectively. Both effects, uniform and differential, were considered independently and simultaneously to explore the most adverse effect. For consistency with the previous results, two values were considered for the initial bridge temperature, T_0 : (i) 15°C in accordance with the stipulated by EC1 in the absence of more precise information and (ii) 22°C , which was the average temperature when the viaduct spans were restrained from deforming freely. Adding the effects due to the time-dependent and the temperature – case A and case B – the values for the measurements that are expected at the end of the viaduct lifetime are presented in Fig. 14. The trends obtained for the concrete deformations with the EC2 models are 55 % to 70 % higher to the ones obtained with the fitted models, which was already expected considering the results

presented in Fig. 8, Fig. 10 and Fig. 13. As regards the temperature effect on the concrete deformations, and considering that the most probable situation corresponds to the simultaneous action of the uniform and differential temperature components, the most adverse effect is achieved when the differential temperature component acts as the main effect. The sensors positioned near the top face exhibit the highest contractions in the summer while the highest expansions are achieved for the winter season. On the other hand, the sensors positioned near the bottom face exhibit highest contractions in the summer while the highest expansions are detected for the winter season. Detailing the results presented in Fig. 14, the concrete deformations near the bottom face at the mid span sections P1P2 and P2P3 exhibit higher sensitivity to temperature with values ranging between $-25\mu\epsilon$ to $+89\mu\epsilon$ for case A and $-33\mu\epsilon$ to $+107\mu\epsilon$ for case B. The remaining concrete deformations exhibit variations between $-30\mu\epsilon$ and $+36\mu\epsilon$ for case A and $-39\mu\epsilon$ and $+32\mu\epsilon$ for case B. These values are coherent with the seasonal variation of the sensors measurements presented in Fig. 13.

Concerning the bearing displacement (BD) at sections TP1 and TP2 (see Fig. 4), the most adverse effect is achieved when the uniform temperature component acts as the main effect. The results presented in Fig. 14 show that the displacements are approximately equal for both sections. The slight difference, in trend and variations caused by temperature, are explained by the higher stiffness of the viaduct over the railway zone (see Fig. 3), which causes a shift of the mass-centre of the viaduct to the side of section TP2, in relation to the half length of the viaduct. Consequently, higher displacements in the bearing of section TP1 are expected. However, in this case the temperature effect has the same influence, or more, as the time-dependent effects because the unrestrained thermal deformations are included, contrarily to what happens for the concrete deformations. It should be remembered, as previously referred in the description of the monitoring system, that the measurements concerning the concrete deformations were corrected by eliminating the effect of the free thermal deformation of the sensor and concrete.

Based on the results herein presented, expansion and contractions movements are expected because of the reduced deformations due to creep and shrinkage in case B, in opposite to the predictions based on the European standard rules only (case A). These results are consistent with the ones presented in Fig. 13-a, where it is clearly visible the seasonal expansion and contraction movements due to temperature, while the results

obtained with the fitted EC2 models shows that the displacements due to shrinkage and creep deformations of concrete are significantly lower. Analysing the results presented in Fig. 14, the bearing displacement at section TP1, BD-TP1, should present a trend value after 100 years of -101 mm, value that can range between -45 mm and -135 mm due to temperature effects. However, the study herein presented shows that the expected trend value is -37 mm (63 % less) and ranging between $+5$ mm and -76 mm due to temperature effects (amplitude 21 % less, which is in accordance with the difference obtained for the thermal dilation coefficient of the concrete).

It is worth mentioned that the initial bridge temperature is a determinant aspect. Higher values of the initial bridge temperature will lead to higher contractions and smaller expansions due to temperature variation, which justifies the different amplitudes for the positive and negative sides of the range values obtained for both cases. Hence, the reviewed predictions, based on the fitted models of the EC2 for shrinkage and creep, are information more realistic and reliable, than the ones obtained based on the standard values prescribed on the regulation guidelines. Therefore, they should be preferred to be used in the maintenance and surveillance of the viaduct. Even though, due to the complexity of the problem, the predictions herein presented should be periodically checked. For that, the numerical model should be updated as the database increases, which will promote the confidence in the obtained results.

5. Conclusions

1. The presented work describes the analysis strategy that was developed to calculate the long-term behaviour of a concrete viaduct. A finite element model was employed, taking into account the effective properties of the structural materials, the actual sequence of construction and the influence of concrete time-dependent effects. A good agreement between the numerical and the experimental results were observed. However, this was possible thanks to the data collected during the execution of the structure, which is not available at the design stage of each new structure. The sensors measurements revealed to be an important element to get an optimized model, with benefits in future updating. Therefore, this type of data becomes more and more useful in the surveillance and maintenance of the structure.
2. The analysis performed since the beginning of the construction indicates that, even after the prestressing, the span weight is partially supported by the movable

scaffolding system. Moreover, the results show that the stress level at the bottom face increases approximately 24 %, although, the limit of $0.45 \cdot f_{ck}$ was never exceeded.

3. The EC2 models overestimate the deformations due to shrinkage and creep of the employed concrete in the viaduct. The superplasticizers and pozzolanic materials used in concrete mix, whose effect cannot be accurately reproduced by code formulae can explain these deviations.
4. Even with the EC2 models for shrinkage and creep fitted to experimental data, some relevant differences are observed between the sensors measurements and the numerical model. Two possible explanations can be pointed out: (i) Comparing the notional size values of the deck girder with the prisms, they are substantially different and therefore, the extrapolation of the shrinkage and creep of the deck girder based on the measurements of the concrete prisms, is more prone to errors. (ii) The sensors near the top face of the deck girder are exposed to particular conditions because of the bituminous layer that increases the concrete temperature and reduces the concrete humidity. It was also observed that the higher is the sensitivity of the sensor measurements to the temperature variation, better is the adjustment with the numerical model results.
5. The effect of the relaxation of the prestress steel is small, with a contribution lower than 2.0% regarding the long-term response for the structure lifetime. The low relaxation of the applied steel and the stress level of approximately of $0.71 \cdot f_{puk}$ justify the small influence.
6. As regard the long-term results, it seems to be difficult to get, simultaneously, the same results quality for the construction and service life of the viaduct. This limitation can be explained by the use of a shrinkage model that attempts to fit to measurements that exhibit different trends during the construction and for the service life.
7. The environmental effects, namely those due to the temperature, are perfectly noticeable in the sensors measurements. The concrete deformations are more sensitive to the differential component, namely the sensors near the bottom face of the mid spans. As regard the bearing displacements, they exhibit higher sensitivity to the uniform component. In this case, the temperature effect has the same importance as the time-dependent effects because the unrestrained thermal

deformations are included in the sensors measurements, in opposite to the concrete deformations measurements herein presented.

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Fig. 1. Lezíria Bridge – construction stage at May 2007 (© TACE).

Fig. 2. Construction views of the north approach viaduct.

Fig. 3. Elevation of the V2N viaduct.

Fig. 4. Identification of the monitored cross sections.

Fig. 5. Layout of the vibrating wire strain gauges positioning at cross section: a) support zone, b) mid-span.

Fig. 6. FEM of the viaduct (DIANA output): a) general view, b) detailed view of span P2P3.

Fig. 7. Compressive strength evolution.

Fig. 8. Shrinkage strains of the deck concrete.

Fig. 9. Shrinkage curves for the structural elements of the viaduct.

Fig. 10. Creep strains of the deck concrete.

Fig. 11. Load test results.

Fig. 12. Results for the instrumented section P2P3 during the construction.

Fig. 13. Long-term results.

Fig. 14. Predictions for the viaduct lifetime.

Table 1. Time history of the V2N viaduct construction.

	2005				2006				2007			
	1 st	2 nd	3 th	4 th	1 st	2 nd	3 th	4 th	1 st	2 nd	3 th	4 th
Piles												
Columns												
Deck												

Table 2. Compressive cylinder strength of concrete at 28 days (MPa) – statistical data.

	Minimum Value	Maximum value	Mean value	Standard deviation	Coefficient variation
Piles (C35/45)	46.2	50.2	48.9	1.4	2.9%
Piers (C30/37)	38.1	46.3	43.7	3.0	6.9%
Deck (C35/45)	45.5	53.1	49.4	2.0	4.0%

Table 3. Mechanical properties of concrete - average values.

	f_{cm} (MPa)	E_c (GPa)	s
Piles	48.9	37.2	0.20
Piers	43.7	35.9	0.18
Deck	49.3	37.3	0.20

Table 4. Subgrade reaction module, k_s [24].

Soil description	Clay deposits	Sludge	Muddy fine to medium sand	Medium sand	Clay silt
Deepness (m)	0 to 2	2 to 6	4 to 6	6 to 8	> 6
k_s (MN/m ³)	4 to 8	1 to 2	7 to 3	8 to 2	90 to 120