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Assessment and management of concrete bridges supported by monitoring data-based Finite Element modelling

Helder Sousa¹, João Bento², Joaquim Figueiras³

ABSTRACT

The long-term assessment of large and complex civil infrastructures, such as prestressed concrete bridges, based on Finite Element (FE) modelling remains a challenging task. The construction process, the influence of erection methods, the characterization of concrete properties, geometric accuracy and environmental conditions are key factors involved in the development of robust FE models. Data collected using permanently installed monitoring systems is the most reliable strategy to improve such assessments. Indeed, the availability of monitoring data is increasingly being used in the validation of design assumptions, updating of FE models and safety factors.

In this work, the long-term behaviour of a long segmental bridge built in Portugal – Lezíria Bridge – is evaluated using FE model-updating. The combination of several factors, including: (i) the bridge's scale, (ii) the monitoring database, (iii) the comprehensive scanning of important characteristics of the bridge and (iv) the FE modelling approach, makes this case study unique. Although the sensor trends are satisfactorily predicted, extrapolation of shrinkage and creep models, the influence of interior and exterior environments and thickness variations of the structural elements are identified as areas for further research.

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Keywords: Structural monitoring, box girder, concrete bridge, finite element modelling, shrinkage and creep.

INTRODUCTION

The long-term assessment of large and complex civil infrastructures, such as prestressed concrete bridges, remains a challenging task. Finite Element (FE) modelling is one of the most widely used and reliable assessment methods for such structures. These models are able to capture satisfactorily the short-term performance, e.g. load tests; however, long-term performance is not so straightforward. This is further hampered by new bridge designs, and advances in structural materials and construction methods.

The balanced cantilever method is widely used in bridge construction for spans ranging from 100 to 300 m (Takacs 2002), and mainly when the use of fixed formwork systems is not possible e.g. over deep valleys, wide rivers, traffic yards or soft soil. In recent years, much interesting research has shown that the long-term predictions of these types of bridges can differ significantly from field observations. In Robertson's study (Robertson 2005), the significant differences noticed between design and observed long-term vertical deflections was mainly attributed to increased creep and shrinkage. Indeed, Goel et al. (Goel *et al.* 2007) show, based on the RILEM data bank, that none of the shrinkage and creep models is able to offer reliable predictions. Recently, Bažant et al. (Bažant *et al.* 2012) show, in the re-assessment of the collapse of the Koror-Babeldaob Bridge, Palau, that excessive long-term deflections are also due to differences in the rates of shrinkage and creep associated with different thicknesses of slabs in the box cross section. In addition, advanced 3D models are advocated by the authors as the most suitable to predict the long-term performance of these types of

bridges. Other authors (Malm and Sundquist 2010, Křístek and Kadlec, 2013) also share this conclusion. In addition, environmental conditions, i.e. variations in temperature and humidity, are also identified as key parameters affecting the accuracy of long-term assessments (Barsotti and Froli, 2000; Roberts-Wollman *et al.* 2002; Li *et al.* 2004).

However, some issues in the aforementioned studies still require further research. In Robertson's study (Robertson 2005), short-term test data was used to obtain long-term predictions for shrinkage and creep. However, Bažant (Bažant *et al.* 2001) show that reliable determination of the final shrinkage might require measurements up to 5 years. Indeed, none of the aforementioned studies show evidence of field data available for the period of analysis. In addition, simplifications in the materials characterization and cantilever process are normally forced due to the lack of field data: Bažant *et al.* (Bažant *et al.* 2012) had to deal with limited data concerning the concrete properties and environmental histories for the Koror-Babeldaob Bridge; Malm and Sundquist (Malm and Sundquist 2010) simplified the cantilever process by introducing the post-tensioning right at the beginning of the subsequent segment. On the other hand, although advanced 3D models might be feasible for bridges with a smaller number of spans which benefit from geometric symmetries, this might be impractical for others with a higher number of spans and without symmetries. Finally, the aforementioned studies focussing on the effect of environmental conditions are mainly related to thermal gradients, whereas the assessment of the effect of both inside and outside temperature and humidity conditions, in relation to the box girder, is not addressed in any of these studies.

Hence, to reduce uncertainties and improve the reliability of predictions, all significant issues with direct influence on bridge behaviour must be taken into account during assessment, if field data exists.

The monitoring of the recently built 970 m long Lezíria Bridge in Portugal (COBA-PC&A-CIVILSER-ARCADIS 2005a; Sousa *et al.* 2011) provides a unique opportunity to better understand the long-term behaviour of segmental bridges and clarify some of the aforementioned limitations. The combination of several factors, including: (i) the bridge's scale, (ii) the monitoring data collected since the beginning of construction, (iii) the comprehensive scanning of real data related with materials, geometry and loading and (iv) the FE modelling approach based on a full model of the bridge including a detailed time-step analysis from the beginning of construction, makes this case study unique. A detailed discussion about the bridge's behaviour is performed by highlighting how important it is to understand the measurement pattern during: (i) the construction stage, (ii) the load test and (iii) the bridge's long-term behaviour. This kind of discussion is quite rare in the literature mainly due to the lack of such comprehensive monitoring databases as the one herein presented. Shrinkage and creep are extensively discussed in order to improve the quality of long-term predictions and their accurate assessment by outlining the main reasons for the observed errors. Finally, based on the results presented in this study, conclusions are drawn, which reflect the benefits of an integrated framework consisting of both FE modelling and structural monitoring, with ultimate aim of supporting bridge management.

CASE STUDY

Description of the structure

The main bridge of Lezíria Bridge is a 970 m prestressed concrete structure, with eight spans of 95 + 127 + 133 + 4×130 + 95 m length supported on two transition piers and seven piers founded on pilecaps over the riverbed. The foundation of each pier is placed on eight circular piles of 2.2 m diameter, except for piers P1 and P2 (which sets

the navigation channel) with 10 piles each. The piles are constructed using permanent metallic casing that reach the Miocene layer at approximately 40 m deep (Figure 1). The pilecaps are built with precast concrete caissons, a permanent formwork, with $5.0 \times 11.0 \times 16.5$ m (height $\times$ width $\times$ length), excepting P1 and P2 that have $8.0 \times 11.0 \times 22.0$ m. A set of four concrete walls with 1.20 m thickness is placed on the pilecaps of each pier. The width and height of the concrete walls range from 3.90 m to 7.50 m and from 13.48 m to 16.66 m, respectively.

The bridge girder, which is monolithically connected to the piers, is a box girder of variable inertia with a height ranging from 4 m to 8 m (Figure 2), excepting at TPN, P6, P7 and TPS (Figure 1). The construction of the box girder core started with a so-called *segment 0*, which later was used to support/position a movable scaffold required by the segmental construction – balanced cantilever method. The construction of the end-spans was partially supported by a formwork placed on the soil surface. The closing segments were built by using one of the movable scaffolds. In the second stage, a metallic truss was used to support the console slabs during their construction with a formwork suspended at each extremity. Finally, the service walkways were built, the bituminous layer poured, and the expansion joints, exterior safety barriers, railings and border beams were placed. The bridge construction started in January 2006 and took approximately 18 months to be complete (COBA-PC&A-CIVILSER-ARCADIS 2005a).

Description of the monitoring system

The instrumentation is spread along several cross-sections of the bridge; however, two zones were comprehensively instrumented since the construction phase. In this context, the focuses of this work are: (i) the first three spans between piers TPN and P3 – Zone 1 and (ii) the last two spans between piers P6 and TPS – Zone 2 (Figure 1).

Most of the sensors, which are vibrating wire strain gauges (CD), were installed on two types of sections located at: (i) 11 m apart pier axis and (ii) mid-spans. Generally, each section has six sensors installed, except for sections P1 and P1P2 in which two additional sensors were installed at the extremities of the console-slabs (Figure 2). All CD were aligned with the longitudinal axis of the bridge. These sensors incorporate an internal thermistor that was used to measure the concrete temperature (CT) in sections P1 and P1P2. The strain measurements were corrected by removing the free thermal deformation of both the sensor wire and the concrete. A thermal dilation coefficient of $11 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ (given by the manufacturer) and $7.9 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ (experimentally evaluated in climatic chamber tests) were considered, respectively.

Ten concrete prisms of dimensions 15 cm × 15 cm × 55 cm with two long unsealed faces were used to measure the time-dependent deformations of concrete: six for shrinkage and the remaining four for creep. Moreover, two shrinkage prisms were cast with concrete used on Zone 2, whereas the remaining prisms, shrinkage and creep, were cast with concrete used on Zone 1. Similar curing conditions were assured for the prisms in order to obtain representative measurements of the concrete poured on both of these Zones.

Finally, bearing displacements (BD) are measured at the supports TPN, P7 and TPS with LVDTs, rotations are also monitored at sections P1, P2 and P7 with electric inclinometers and the vertical displacements (VD) of all mid-span sections are also monitored with optic sensors supported by a hydrostatic levelling system (Figueiras *et al.* 2010). Temperature and relative humidity of both inside and outside surroundings of the box girder were measured at section P1, as well as corrosion and accelerations. A comprehensive description of the monitoring system can be found elsewhere (Sousa *et al.* 2011).

Finite Element Analysis

General considerations

A non-linear finite element analysis was performed by using the general-purpose finite-element code DIANA (Manie 2008). Structural discretization was carried out using beam elements of approximately 1 m to 2 m long, depending on the structural component type, i.e. piles, piers and girder. The ordinary and prestressed reinforcements were modelled using embedded reinforcing elements, whose deformation is calculated from the displacement field of the concrete finite-element in which they are embedded. Both instantaneous and time-dependent prestress losses are automatically computed. The piles-soil interaction was also modelled with elastic springs. A phased analysis with 105 stages was performed to simulate the real chronology observed during the construction (TACE 2007). For each new stage, new elements were added/removed and/or the support system was updated. Concerning large-scale bridges, errors are inevitable during the FE model implementation (Catbas *et al.* 2007), and therefore, CAD tools were specifically developed and used during the scanning of the drawing pieces in order to mitigate potential errors, and hence reduce the computational time (Sousa 2012).

Structural modelling

All structural components were defined based on the final project drawings (COBA-PC&A-CIVILSER-ARCADIS 2005a) and represented by their axis (Figure 3-a). Two overlapped alignments of beam elements were modelled, with the same displacement field, to simulate both the box girder core and the console slabs, in order to take into account the phased construction of the bridge girder.

Layers of ordinary reinforcement were modelled along the edges and axis of the beam elements in order to take into account their restrained effect in time-dependent deformations of concrete. The embedded prestressing cables of the bridge girder were precisely modelled with parabolic elements (Figure 3-b). As far as the external prestressing cables are concerned, truss elements were used and connected to the girder beam elements through dummy elements with high moment of inertia. Finally, the movable scaffolding systems used in the balanced cantilever construction was also modelled with beam elements; these were assumed to be connected to the girder ends.

Overall, the numerical model has 1804 beam elements, 633 truss elements, 5106 reinforcement elements (including ordinary reinforcements and prestressing cables), 248 spring and 16 supports.

Concrete modelling

The reliable prediction of structural behaviour requires the accurate modelling of the material properties, especially those related to concrete. To this end, the evaluation of the mechanical properties of concrete was based on a set of measurements collected from early ages, i.e.: (i) compressive tests on 150 mm cubes performed during the construction (TACE 2007) and (ii) measurements taken from concrete prisms to evaluate shrinkage and creep deformations.

The models of the European Code Eurocode 2 (EC2) were used to describe the time-dependent properties of concrete (European Committee for Standardization 2004), with the parameters of the EC2 models defined based on the aforementioned experiments. Moreover, the concrete properties for each structural component were detailed in the FE model in order to improve the model accuracy, especially during the construction stage.

Evolution of concrete compressive strength

The evaluation of the variation of the concrete compressive strength with time is crucial information for long-term analysis, due to its correlation with the evolution of the concrete modulus of elasticity. The compressive strength at a given age, $f_{cm}(t)$, is given in the EC2 by Eq. (1), where t represents the concrete age in days, s is a cement-hardening coefficient, and f_{cm} is the mean value of the concrete compressive strength, at the age of 28 days.

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

The characterization of the compressive strength of all structural components was exhaustively scrutinised. Due to the fact that concrete cubes were used, it took 82 % of the observed values to obtain the corresponding cylinder compressive strength's, $f_{cm,cyl}$, as recommended by EC2. Regarding the parameter s , this was determined by a curve fitting procedure that minimized the mean square error between the test results at different ages and the Eq. (1).

Modulus of elasticity

The determination of the tangent modulus of elasticity, E_c , was based on the compressive strength, by means of Eq. (2), where $f_{cm,cyl}$ represents the mean value of the concrete cylinder strength at the age of 28 days.

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

The time-dependent variability of the concrete elasticity modulus correlates to the time variation of the compressive strength (determined in the preceding equation) and is

given by the following equation according to EC2. Table 1 summarizes the average values of the parameters previously discussed for each type of structural component.

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

Shrinkage

The total shrinkage strain, ϵ_{cs} , is set in EC2 by two parts: the drying shrinkage strain, ϵ_{cd} , and the autogenous shrinkage strain, ϵ_{ca} (Eq. (4)). Both mathematical models are expressed by a multiplicative model with a nominal coefficient, $\epsilon_{c,\infty}$ and a time factor, $\beta_s(t)$, where t is the time (in days) since drying begins, t_s (Eqs. (5) and (6), respectively).

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

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

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

Additionally, $k_{cs,0}$ and $k_{cs,t}$ parameters were added to Eq. (5) so that the drying shrinkage model could be scaled and shaped to experimental results obtained from the concrete prisms. This was determined by a curve fitting procedure that minimized the mean square error between the test results at different ages and the Eq. (5) (Santos 2002).

The autogenous shrinkage was not considered in the fitting problem since: (i) it mainly occurs during the early days after casting (European Committee for Standardization 2004) and (ii) the FE model focuses on the long-term behaviour of the bridge. In other words, its effect was initially removed by subtracting a quantity

expressed by Eq. (6) to the measurements, and after the fitting problem was solved its contribution was restored.

The concrete prisms for shrinkage were positioned inside and outside of the box girder, in order to take into account the effect of the different surrounding environments in the time-dependent deformations of the box-girder concrete. However, the influence of each environment is still difficult to assess, and at the present state of knowledge, an accurate quantification of their influence is not yet possible (Santos 2007). However, and for simplicity, their effects were taken proportional to the cross-section perimeter exposed to each environment (Sousa *et al.* 2012). Consequently, the time-dependent deformations of the girder concrete are calculated based on 30 % and 70 % of the prisms placed inside and outside the box girder, respectively. Additionally, an average temperature of 18.8 °C and 16.1 °C, and a humidity of 51.8 % and 64.0 % were set, based on measurements, for the interior and exterior environments, respectively. The assumption of average values for temperature and relative humidity is acceptable without significant errors for long-term analysis (Barr *et al.* 1997).

Figure 4-a shows the shrinkage results for Zone 1 (Figure 1) where it can be seen that the exterior prisms shrink more than the interior ones. The different initial patterns, mainly during the first 250 days, and the parallel patterns after that period might be explained with the following reasons: (i) the positioning of the exterior prisms over the top slab of the deck girder during the construction led to direct exposure to the sun which might have accelerated the shrinkage evolution; (ii) after the construction ended, the exterior prisms were placed on the top of the transition piers and under the deck girder i.e. sheltered from the sun, and therefore, the environmental conditions changed by slowing the shrinkage evolution. Similar results are observed for Zone 2.

Overlapping the theoretical models of EC2, the results are not significantly different from the shrinkage measurements (Figure 4-a). Indeed, the shrinkage deformations from both zones are nearly identical ($k_{cs,0} = 0.91$ for Zone 1 and $k_{cs,0} = 1.00$ for Zone 2), except for the evolution during the first months ($k_{cs,t} = 0.90$ for Zone 1 and $k_{cs,t} = 0.41$ for Zone 2). The different concrete mixing plants used for each zone and the fact that pumped concrete was used are additional factors to justify these differences.

The notional size, defined in EC2 as $2 \cdot A_c / u$, where A_c is the concrete cross-sectional area and u is the perimeter of that part which is exposed to drying, of the structural components of the bridge ranges from 400 mm to 1500 mm, which differs significantly to the shrinkage prisms that are 150 mm. Therefore, the shrinkage curves for the structural components were obtained based on Eq. (4) by considering the $k_{cs,0}$ and $k_{cs,1}$ values computed for the shrinkage prisms as well as the respective concrete properties and notional size. Moreover, the thickness differences between the bottom slab, web and top slab constitute another key issue, which influences the long-term behaviour of this type of bridge (Kristek *et al.* 2006; Bazant *et al.* 2008; Malm and Sundquist 2010). Hence, three zones were identified for each cross-section – bottom slab, webs and top slab – to allow for different shrinkage patterns under the same cross-section. The computed shrinkage curves show that, after 1250 days, the deformations of the bottom slab near the piers ($190 \mu\epsilon$) are expected to be approximately half that of those near the mid-spans ($370 \mu\epsilon$). Overall, a set of 333 shrinkage curves were used in the FE model.

Creep

The creep deformations of concrete, $\varepsilon_{cc}(t, t_0)$ at a generic time t for a constant applied compressive stress at age t_0 is given in EC2 by Eq. (7), where $\varphi(t, t_0)$ is the creep coefficient and $\varepsilon_c(t_0)$ is the instantaneous deformation due to the mentioned compressive stress. The creep coefficient is given by Eq. (8) where φ_0 is the notional creep coefficient and $\beta_c(t, t_0)$ is a function to describe the development of creep with time after loading.

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

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

In the same way as for shrinkage, additional parameters, $k_{cc,0}$ and $k_{cc,t}$, are added in Eq. (8) so that the creep model could be adjusted to experimental data collected from the concrete prisms (Santos 2002), as well as the effect of inside and outside environments on creep deformations also being taken into account.

Figure 4-b shows that the interior prisms have more creep deformations than the exterior ones. The same pattern of the shrinkage prisms (Figure 4-a) is observed, i.e. a different initial pattern during the first 250 days, after which they evolved in parallel. However, the interior measurements are higher in this case and, therefore; it is difficult to explain using the same reasons previously explained for shrinkage. The applied load is another important parameter that can influence creep and inaccuracies in the applied load might lead to additional bias. Nevertheless, this cannot explain the similar pattern observed after construction. Thus, based on these results, it can be concluded that the construction conditions constrained the different creep patterns.

Figure 4-b shows that the fitted EC2 model leads to a more realistic interpretation whereas the overestimation committed by the original EC2 model is clearly visible.

Additional to the reasons discussed for shrinkage, the effect of the superplasticizers and pozzolanic materials employed cannot be accurately predicted using code formulae and, therefore, some bias associated with the EC2 models is expected (Bazant 2001). Finally, the creep curves for the structural components were obtained using a procedure similar to the one adopted for the shrinkage case.

Prestressing cables, soil and loading

A thorough scan was carried out to characterize the mechanical properties of the employed prestressing steel (TACE 2007). A set of average values was calculated for each group of cables based on the manufactures specifications. The relaxation class 2 was adopted, taking into account the low relaxation of the prestressing cables and were placed inside flexible metal ducts. In addition, the recommendations of Model Code 2010 were followed for the wobble coefficient K and the coefficient of friction, μ (FIB Commission on Practical Design 2010).

The piles–soil interaction was modelled using elastic springs with their behaviour described by the Winkler model. The spring stiffness is taken proportional to the influence area of each spring, A_{inf} , and the subgrade reaction module, k_s (COBA-PC&A-CIVILSER-ARCADIS 2005b).

In relation to the loading conditions, the following were considered: (i) self-weight of the reinforced concrete, γ_c ; (ii) self-weight of the movable scaffolding systems based on the equipment specifications, F_{ms} (TACE 2007); (iii) forces applied to the prestressing cables based on the elongation measurements (TACE 2007), f_{PE} ; (iv) dead loads with respect to the bituminous layer, border beams, walkways and safety barriers, p_{dl} ; (v) fully loaded trucks used in the load test, F_t (Table 2).

RESULTS AND DISCUSSION

Load test

Measurements collected during live load tests are the most suitable strategy to initiate the FE model validation, due to the fact that the loading is known quantitatively and can easily be controlled.

Figure 5 depicts the time-series obtained for both numerical results and measurements concerning the load test. Eight Load Cases (LC1 to LC8) explored the maximum span deflection by using three alignments of six trucks. Figure 1 details the trucks' positioning and load magnitude. Overall, a good correlation is observed between the numerical results and the respective field measurements during the observation period in relation to both pattern and amplitude. The following observations can be made with regard to the maximum amplitude: (i) the higher displacement is observed in section P6P7 (28.5 mm against the measured 26.7 mm), while the lowest occurs for section PTNP1 (21.3 mm against the measured 21.2 mm). Errors range from -4.1% (P7PTS section) to $+6.6\%$ (P6P7 section); (ii) the rotation results in Figure 5-b show that deviations are slightly higher, with a maximum value of 9.4% (P1 section) and 10.9% (P2 section). However, these sections are subjected to higher shear forces due to their location near the piers and the LCs which lead to cross-section warping and therefore, additional deviations are expected; (iii) concerning the section curvatures (calculated from the measured concrete deformations), almost a perfect matching is attained for sections near piers (P1 and P2), while slight differences are observed for the mid-span sections (TPNP1, P1P2 and P2P3), with a maximum difference of -5.8% for section P2P3 (Figure 5-c, d). Therefore, and based on the load test results, the adopted

finite elements discretization and simulation of the actual supports proves to be suitable to simulate the structural behaviour of the bridge.

Construction assessment

Concrete pouring

Generally, the collection of strain measurements was initiated prior to concrete pouring. However, the zero-reference must be carefully set because during this period, the concrete transforms from a mass to a structural material. Measurements indicated that the concrete temperature stabilizes 5 to 7 days after concrete pouring. Due to the fact that the aim of this work is to predict the long-term performance of the bridge, concrete hydration and temperature fluctuations were not considered. The assessment of these effects requires a more refined model, which is out of the scope of this study. In this context, and based on the collected measurements, the zero-reference for strain measurements was set 7 days after concreting.

Segmental construction by the cantilever method

Typically, the construction of one segment of the bridge girder took 7 days with the: (i) positioning of the movable scaffolding, (ii) reinforcement placement and concrete pouring, (iii) tensioning of the prestressing cables, (iv) release of the movable scaffolding and moving forward.

Figure 6 shows the concrete deformations (CD) during the construction from sections P1 and P7, with the measured values being correctly predicted by the numerical model. Slightly different patterns are observed for the results in the top layer during the construction of the first segments, which can be explained by the use of different movable scaffolding systems in both zones. Actually, 13 segments of 3 m to 5 m long were built on pier P1 whereas 9 segments of 6 m to 7 m long were built for the

cantilever construction on pier P7 both with different prestressing schemes, which explain the different patterns. These effects are not so clear in the bottom layer due to the positioning of the console prestressing cables in the top slab.

The results also show that sections P1 and P7 are practically uniformly compressed (small bending effect), which is explained by the prestressing scheme that balances the opposite bending effect caused by the weight of the deck girder. The effect of temperature on the measurements is another relevant issue, especially for the top layer due to direct exposure to the sun. The higher temperature amplitudes in this layer justify the different pattern when compared to the ones observed in the bottom layer.

The stress level at the end of the construction is quite similar in sections P1 and P7, despite the different segment lengths and prestressing schemes. The concrete stress in the bottom layer is -8.1 MPa and -8.3 MPa whereas in the top layer is -11.1 MPa and -11.9 MPa for sections P1 and P7, respectively. These FE results reveal that the serviceability limit for the compressive stress on concrete was not exceeded during construction (European Committee for Standardization 2004).

Long-term behaviour

Deck girder sections near piers

Regarding the long-term behaviour, Figure 7 extends the results presented in Figure 6. Data is only available until August 2010 for sections P7 and P6P7. The numerical results shown in Figure 6 are based on the fitted EC2 models (Figure 4); to better inform the discussion, the numerical result obtained using the original EC2 models are also shown. Overall, a slight overestimation is observed by the numerical results based on the EC2 models whereas those obtained with the fitted models show a better correlation with the measurements. Even so, the trend of the measurements seems better

398 predicted by the results obtained using the original EC2 models. This can be explained
399 thus: (i) although the fitting result for shrinkage model did not imply a significant
400 change, the same cannot be assumed for creep, for which a significant improvement in
401 the trend pattern was achieved with the fitted model (Figure 4); (ii) the cross-section
402 discretization in three different areas allows different velocities for shrinkage and creep
403 evolution in the same cross-section and, consequently, the FE model has higher
404 flexibility. Nevertheless, the notional size of these three areas is significantly higher
405 than that of the concrete prisms. This means that the extrapolation of the shrinkage and
406 creep curves for the structural components might be inadequate, and it might become
407 critical as the notional size increases. Even so, if the results are analysed in a long-term
408 perspective, the trends of the measurements can be well predicted for both cases.

409 The temperature effect on CD is also worth discussing. Firstly, the decrease in the
410 amplitudes on the daily variation of the CD in the top layer can be explained by the
411 bituminous layer placed at the end of construction. Secondly, the lower sensitivity of the
412 CD of section P7 to seasonal variations in temperature, particularly in the top layer, can
413 be explained by the pier-deck connection. Contrary to the monolithic connection
414 between pier and deck at P1, the connection in P7 is provided by roller supports, which
415 allow a higher parcel of deformations due to temperature variation taking place in this
416 section.

417 Some measurements show patterns which are not correctly predicted by the FE
418 model. Particularly, some measurements in the alignment 3S show higher trends which
419 can be explained by the positioning of concrete safety barriers exactly above this
420 alignment (Figure 7-a). Furthermore, a concreting operation took place, in a second
421 stage, to allow the positioning of the barriers. As a result, additional deformations were
422 induced in the top layer of the box girder due to shrinkage of the second stage

concreting. In other cases, some sensors were affected and potentially damaged due to the grinding of the concrete surface for the safety barrier's positioning, as occurred in section P7 (Figure 7-b).

Deck girder sections at the mid-span

The results for the mid-span sections P1P2 and P6P7 are shown in Figure 8. Similar conclusions to those presented for sections P1 and P7 can be drawn. Moreover, the results for these sections are slightly better than those near the piers, which can be explained by the thickness difference between the bottom and top slab (see Figure 1). Actually, the thicker the element, the larger the deviations for the extrapolated shrinkage and creep curves (based on the concrete prisms). Nonetheless, if the results are analysed in a long-term perspective, the trends of the measurements are well predicted in both cases, as in cases of sections P1 and P7.

Bearing displacements

The evolution of the bearing displacements is particularly important for the validation of the FE model because it reflects the global horizontal behaviour as well as allowing validation in relation to the horizontal forces. Figure 9 shows the results for sections TPN and TPS (see Figure 1). The overestimation predicted by the original EC2 models is confirmed. In this case, the differences are greater when compared to the case of the concrete deformations. Conversely, the results obtained based on the fitted EC2 models predicted the measurements more realistically. The aforementioned bearing displacements reflect the global response of the bridge whereas the concrete deformations are local information, which might justify the greater differences observed for the displacements.

CONCLUSIONS

- The analysis strategy to compute the long-term behaviour of a box girder bridge built by the balanced cantilever method is herein fully exposed by considering the following: (i) rigorous scanning of the bridge's geometry, (ii) in-situ properties of the structural materials, (iii) real loads, (iv) mechanical properties of the foundation soil and (iv) real construction sequence. Although recent research has claimed that advanced 3D models are the most suitable option, it is shown, based on the results herein presented, that it is possible to build robust 1D FE models for large-scale bridges such as in the case of Lezíria Bridge for the first 5 years. However, this was possible thanks to a unique combination of several factors, which is not available at the design stage of a new structure. Even so, further research is suggested, mainly through the development of an optimized advanced 3D model and by quantifying differences between both versions envisioning the bridge life cycle.
- Concerning the load test, a good correlation is observed during the observation period in relation to both pattern and amplitude. In particular, differences less than 6.6 % are observed for the maximum vertical displacements of all mid-span sections. In addition, the numerical results obtained for concrete deformations show high conformity with the measurements for the construction period. In this context, it is possible to state that the numerical model predicts with sufficient accuracy the bridge's performance under static loads.
- With respect to long-term performance, the sensor trends are satisfactorily predicted by the numerical results based on both the fitted and original EC2 models. This might be useful information to support bridge management decisions and set more realistic performance criteria. Moreover, the measurements collected from the shrinkage and creep prisms were crucial for obtaining these results.

- 471 • Unexpected behaviour is observed for some CD, which cannot be correctly
472 predicted using the numerical model. This fact is explained by considering the local
473 conditions where the sensors are positioned, for which the global model herein
474 presented is not suitable. This observation reveals the importance of having
475 complete information about what happens during construction in order to have a
476 comprehensive understanding of the collected data.
- 477 • The zero-offset for the CD is a key issue for long-term predictions; however, this is
478 not normally clarified in similar studies. Based on the CD measurements, a period of
479 7 days after concreting is recommended for the zero-offset. Before this date, the
480 concrete hydration thermal effect disables an accurate interpretation of the CD
481 measurements.
- 482 • Despite the rigorous model update followed in the FE analysis, some differences
483 were observed between the measurements and numerical results. In relation to the
484 long-term response of the bridge, three main reasons are given for these differences:
485 (i) the extrapolation of the shrinkage and creep curves for the structural components
486 based on the prism measurements being prone to errors, due to the significant
487 difference on the notional size, (ii) the lack of knowledge of the relative importance
488 of interior and exterior environments on the long-term response of the bridge, and
489 (iii) the thickness difference between bottom slab, web and top slab implying
490 different velocities in the shrinkage and creep evolutions, which is very difficult to
491 model. Although the adopted strategies to overcome these issues led to reasonable
492 results, further research is recommended.
- 493 • The results presented in this study improve and validate the use of this modelling-
494 update approach to support management decisions throughout the operational

lifetime of segmental bridges, as long as continuous updating is provided in the future use of collected monitoring data.

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Table 1 – Mechanical properties of concrete - average values.

	f_{cm} (MPa)	E_{cm} (GPa)	s
Piles	50.4 ($c_v^* = 8.7\%$)	37.5	0.23
Piers	56.6 ($c_v^* = 4.4\%$)	38.8	0.25
Deck	55.5 ($c_v^* = 5.2\%$)	38.6	0.26

* coefficient of variation

Table 2 – Main properties of prestressing cables, soil and loading.

Prestressing steel	Soil	Loading
$f_{pum} = 1911$ MPa ($c_v^* = 0.9\%$)	$k_s = 4$ to 8 MN/m ³ (0 to 2 m deep)	$\gamma_c = 25$ kN/m ³ $f_{PE} = 0.71 f_{pym}$ ($c_v^* = 6.6\%$)
$f_{pym} = 1765$ MPa ($c_v^* = 0.9\%$)	$k_s = 1$ to 2 MN/m ³ (2 to 6 m deep)	$p_{dl} = 93.7$ kN/m
$E_p = 196.6$ GPa ($c_v^* = 2.3\%$)	$k_s = 7$ to 30 MN/m ³ (4 to 6 m deep)	$F_{ms} = 570$ to 1127 kN $F_t = 315.9$ kN ($c_v^* = 1.6\%$)
$K = 0.05$	$k_s = 8$ to 20 MN/m ³ (6 to 8 m deep)	
$\mu = 0.19$	$k_s = 90$ to 120 MN/m ³ (> 6 m deep)	

* coefficient of variation

Figure 1 – Elevation of the bridge zones intensively instrumented: a) Zone 1, b) Zone 2.

Figure 2 – Layout of the vibrating wire strain gauges positioning at cross-section: a) 11 m apart the pier axis, b) mid-span.

Figure 3 – FE model of the main bridge (DIANA output): a) overall view, b) detailed view of half-span P1P2.

Figure 4 – Time-dependent deformations of concrete: a) shrinkage, b) creep.

Figure 5 – Load test results: a) vertical displacements, b) rotations, c) curvature of sections near the piers, curvature of mid-span sections.

Figure 6 – Concrete deformations in sections P1 and P7 (construction): a) section P1 – top layer, b) section P7 – top layer, c) section P1 – bottom layer, d) section P7 – bottom layer.

Figure 7 – Long-term results – sections near the piers: a) section P1 – top layer, b) section P7 – top layer, c) section P1 – bottom layer, d) section P7 – bottom layer.

Figure 8 - Long-term results – mid-span sections: a) section P1P2 – top layer, b) section P6P7 – top layer, c) section P1P2 – bottom layer, d) section P6P7 – bottom layer.

Figure 9 – Long-term results – bearing displacements: a) section TPN, b) section TPS.