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NUMERICAL EVALUATION OF THE LONG-TERM BEHAVIOR OF PRECAST CONTINUOUS BRIDGE DECKS

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Abstract

Continuous bridge decks constructed with precast girders undergo significant stress changes caused by the concrete delayed deformations due to creep and shrinkage. These effects must be taken into account in the design of new structures. The validation of the analysis procedures should ideally be carried out through the comparison between the calculations and the results observed in real structures. However, experimental results of the construction and long-term behavior of these structures are scarce. The construction of a major bridge in Portugal has provided the opportunity to monitor one such structure. This paper presents the monitoring campaign and the analysis strategy which was developed to assess the long-term variation of

strains and stresses in precast continuous bridges. The numerical analysis was validated by comparison with the results observed in the real bridge. The consequences of carrying out simplified analyses based on limited information about the concrete properties and the construction sequence are also evaluated.

Introduction

Precast prestressed beams have been widely employed in the construction of modern highway and railway bridges. The elimination of the deck joints by making the girders continuous leads to several practical and economical benefits, such as increased strength and stiffness, improved durability, better driving surface and reduced maintenance costs. However, continuity makes the analysis of the long-term behavior more complex. In fact, the combined effects of concrete creep and shrinkage cause gradual changes in the internal forces and stresses in these composite beams, which behave as simple-span structures when the self-weight and prestress are applied and as continuous beams for the remaining loads.

Several methods have been proposed to evaluate the long-term behavior of this kind of structures. Early studies by Mattock (1961) gave rise to the PCA method (Freyermuth 1969), the CTL method (Osterle et al. 1989), the software RMCalc (McDonagh and Hinkley 2003) and a numerical methodology which included the effect of concrete cracking (Mirmiran et al. 2001). Both the PCA and the CTL methods have been largely employed by practitioners to predict restraint moments (Hastak et al. 2003). However, the application of those methods is restricted to the most common bridge deck typology, in which the continuity is achieved by means of a continuity diaphragm and a reinforced concrete slab. They can not be employed in the analysis of structures constructed with different techniques, for example if the slab is post-tensioned. That difficulty was overtaken by other authors by using numerical methodologies for the time dependent analysis of concrete frames (Ma et al. 1998, Ghali et al. 2002), or general-purpose finite-element software (Kwak and Seo 2000, Marí 2000).

The validation of the aforementioned calculation methods has traditionally been carried out by comparing the numerical results against the results of experimental studies, which were carried out both under laboratory controlled conditions (Mattock 1961; Valdés 1997; Peterman and Ramirez 1998; Newhouse et al. 2008) and under environmental conditions (Miller et al. 2004). However, laboratorial experiments do not accurately reproduce the scale, the environmental conditions, the construction procedures and the loading that occur in real structures. Ideally, the calculation methods should be validated by comparison with results observed in real structures (Cruz and Wisniewski 2004, Barr et al. 2008), but such field data are scarce due to the difficulty in monitoring full-size bridges over long periods of time.

The recent construction of a major bridge in Portugal - the Leziria Bridge - which comprises a 9.16 km viaduct made up of precast beams, offered the opportunity to observe the long-term behavior of a real structure and to compare measurements against the results of numerical calculations. A permanent monitoring system was installed in this bridge and measurements were taken since the beginning of the construction. The data collected in the first 32 months of observation is used in this paper with the following purposes: to evaluate the adequacy of finite element models (considering creep, shrinkage, cracking, relaxation and the construction sequencing) to predict the real behavior of this kind of structures; to assess the consequences of carrying out simplified analysis based on limited information about the concrete properties and the construction sequence.

Firstly, this paper presents the structure and the monitoring system. Then, the strategy implemented for the numerical analysis is exposed. Finally, the more relevant experimental and numerical results are shown and discussed.

Case study

Description of the structure

The south approach viaduct of the Lezíria Bridge is a 9160 m structure whose deck is made up of precast beams. It is divided into 22 elementary viaducts, separated by expansion joints, with length ranging from 250 m to 530 m. This work focuses on the V14S viaduct, which is 468 m long and is composed by 13 continuous spans with a span length of 36 m each (Fig. 1).

The cross section is 29.95 m wide and is composed by 4 precast U-shaped girders (1.75 m high) and a 0.25 m thick slab (Fig. 2). The girders are spaced 7.50 m from centerline to centerline and are prestressed by straight prestressing strands located at the bottom slab. In the region close to the ends of the girders, the prestressing tendons are partially unbonded from the concrete, in order to avoid excessive compression stresses in the bottom slab. The deck slab is made up of precast concrete planks (whose thickness varies between 0.08 m and 0.10 m) monolithically connected to the remaining cast-in-place (CIP) portion of the slab, see Fig. 2.

Girder continuity is provided by a CIP diaphragm, reinforcing steel bars and straight post-tensioning cables in the CIP slab. The continuity diaphragm, which is 2.30 m thick, is monolithically connected to the CIP piers.

The slab is cast in separate phases. Firstly, a portion of slab above the piers is cast simultaneously with the diaphragms. This slab strip extends approximately 6 m for each side of the pier axis. After this concrete has gained the required strength, the continuity post-tensioning is applied. The mid-span portion of the slab is cast at a later stage.

The foundations are provided by circular CIP piles, with a diameter of 1.5 m (as the piers), crossing alluviums with variable constitution, and reaching the maximum depth of 47 m. Above the ground surface, the pier-pile cross section is kept constant up to the cap (Fig. 2). The pier cap provides the necessary room for positioning the precast girders, avoiding the necessity of provisory support systems.

This approach viaduct of the Lezíria Bridge was constructed in 10 months, beginning in February 2006.

Monitoring system

Vibrating-wire strain gauges and temperature sensors were installed in two mid-span cross sections and two cross sections above the pier axes (Fig. 1). In each cross section, measurements were taken in the beams located in both the alignments A and B. According to Fig. 2, the sensors were installed both in the girder and in the CIP slab. In one of the girders, measurements in the three sensors were taken continuously since the start of concreting. In the cases where it was not possible to measure continuously since the time of casting, reference measurements were taken before the start of concreting. The internal thermistor of the strain gauges was used to measure the concrete temperature. 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 of the concrete. For this purpose, the thermal dilation coefficient took the value $11 \cdot 10^{-6}$ in the case of the wire (given by the manufacturer) and $8 \cdot 10^{-6}$ in the case of the concrete (experimentally evaluated in climatic chamber tests).

Shrinkage was measured in four 15·15·55 cm prisms, which have two faces exposed to drying. Two prisms were made of girder concrete and the other two were cast with slab concrete. The prisms were subjected to the same curing procedure that was applied to the structural elements cast with the same concrete. After pouring the concrete of the second-span deck slab, half of the prisms were kept in interior environment (inside the box girder) whereas the others were kept in exterior environment sheltered from rain.

Only one of the strain gauges (located in the CIP slab) was damaged during the construction, which represents a high success rate.

Finite-element analysis

A nonlinear finite-element analysis was conducted by using the general-purpose finite-element code DIANA (Witte 2005). Structural discretization was carried out using beam elements. The elements are numerically integrated along the beam axis and in the cross section. Longitudinally, each beam element has three nodes and is based on an isoparametric formulation. In this analysis, the beam elements are approximately 1 m long. Only one alignment of girders is modeled, as the loading and slab width over each alignment is approximately equal. In this model, both geometry and loading are symmetric across the vertical axis of the girder.

A phased analysis was performed, the finite-element model being changed at every new stage by modifying the connection to the supports, adding new elements or new parts to the existing cross sections. The sequencing adopted in the analysis follows the chronology observed during the construction, which was recorded for future reference. Events occurring in close dates were, for the sake of simplicity, considered to occur simultaneously.

The reinforcement, both ordinary and prestressed, was modeled using embedded reinforcement elements, whose deformation is calculated from the displacement field of the concrete finite-element mesh in which they are embedded. Both instantaneous and time-dependent prestress losses are automatically computed.

Concrete modeling

Obtaining a correct prediction of the structural behavior requires an accurate modeling of the concrete deformation. The constitutive model adopted to describe the concrete behavior allows the consideration of the effects of creep, shrinkage and cracking. In each point, the total strain ε is given by the sum of a concrete strain ε^{co} and a crack strain ε^{cr} . The Kelvin chain model is employed to describe the viscoelastic behavior of concrete owing to creep and ageing effects. Cracking is reproduced through a smeared fixed-crack approach. In the area surrounding the reinforcing bars, the tension stiffening effect is taken into account by modifying the concrete

stress-strain relationship according to the proposal of Figueiras (1983). Outside that area, a stress-strain model with linear softening is employed to describe the crack formation and growth in plain concrete. A comprehensive description of this constitutive model can be found elsewhere (Borst 1991; Witte and Kikstra 2005; Sousa and Neves 2009).

The evaluation of the concrete properties was based on the measurements taken in 4 shrinkage prisms and on the results of compression tests of 150 mm cubes, which were carried out during the construction, at different ages. Moreover, information was taken from the retro-analysis of a simply-supported precast girder, whose deformation was measured since the start of concrete pouring. The expressions proposed by the European code Eurocode 2 (CEN 2004) were adopted to describe the time variation of the concrete properties (modulus of elasticity, creep and shrinkage). Whenever possible, the value of the parameters involved in those expressions was based on the measurements which were mentioned before. For the sake of simplicity, hereinafter Eurocode 2 will be referred to as EC2.

Evolution of concrete compressive strength

The time variation of the concrete compressive strength is an important information for time-dependent analyses, since it correlates to the evolution of the concrete modulus of elasticity. The compressive strength at a given age, $f_{cm}(t)$ is given in the EC2 by:

$$f_{cm}(t) = \exp\left\{s\left(1 - \sqrt{28/t}\right)\right\} \cdot f_{cm} \quad (1)$$

where t represents the concrete age in days, s is a 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. In this work, the parameters f_{cm} and s were determined by a curve fitting procedure, which consisted of minimizing the mean square error between the cube test results (at different ages) and the equation (1). Table 1 presents the values obtained for those parameters.

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 EC2 equation:

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

where $f_{cm,cyl}$ represents the mean value of the concrete cylinder strength at the age of 28 days. Since the cylinder strength was not experimentally evaluated, it was taken as 82% of the observed cube strength. This relation corresponds to the average value of the ratio between the cylinder strength and the cube strength for the same class of concrete, in EC2. The variation of the concrete elasticity modulus with time correlates to the time variation of the compressive strength (determined above), and is given by the following equation according to EC2:

$$E_c(t) = \left\{ \exp \left[s \left(1 - \sqrt{28/t} \right) \right] \right\}^{0.3} \cdot E_c \quad (3)$$

Shrinkage

The time variation of shrinkage, $\varepsilon_{cs}(t)$, was obtained by fitting the results of the four shrinkage prisms (Fig. 3 and Fig. 4) to the following function:

$$\varepsilon_{cs}(t) = \frac{t}{t + S_1} \cdot S_2 + S_3 \cdot \sin \frac{t \cdot 2\pi}{365} + S_4 \cdot \cos \frac{t \cdot 2\pi}{365} \quad (4)$$

where t is the time (in days) since drying begins, S_1 and S_2 are fitting parameters that characterize the development of drying-shrinkage and S_3 and S_4 are parameters that describe the yearly (seasonal) shrinkage variation. Given that both the girder and the slab are exposed to the interior and the exterior environments, shrinkage was taken as a weighted average of the values observed in the interior and the exterior prisms. At this stage of knowledge, it is not possible to accurately quantify the influence of each environment (Santos 2007). For that reason, it was simply

assumed a contribution of the exterior environment equal to 50% and 75% for the girder and the slab respectively (a higher percentage was adopted for the slab as it has a larger surface exposed to the exterior environment).

The data collected in the monitoring of one of the precast beams during the first period of time in which it behaves as a simply supported structure was also used to obtain additional information regarding the delayed concrete deformation. In fact, if it is assumed that the structure is subjected to known forces during this stage, then a retro-analysis can be performed in order to calculate the creep coefficient $\varphi(t, t_0)$ and the shrinkage $\varepsilon_{cs}(t)$ (two unknowns) that correspond to the observed strains on the top and the bottom flanges of the mid-span cross section (two conditions). In this calculation, the age-adjusted elasticity modulus method (Ghali et al. 2002) was applied to describe the concrete stress-strain relationship. This problem has a single solution if a given value is assumed for the ageing coefficient (it took a plausible value, 0.5, as it doesn't significantly influence the results) and the concrete modulus of elasticity (it was given by Equations 2 and 3 and Table 1). Fig. 5 depicts the time variation of the observed strains and the outcome of the retro-analysis. Shrinkage calculated through the retro-analysis is also plotted in Fig. 4, where it can be observed that the results of this analysis are not significantly different from those measured in the shrinkage prisms.

In the FEM analysis, the result of the retro-analysis was used to describe the shrinkage development in the first 40 days after casting (before continuity is established). Afterwards, the shrinkage variation is based on the results of the shrinkage prisms. A comprehensive description of the retro-analysis and the curve-fitting procedure can be found elsewhere (Sousa et al. 2009).

Creep

The results of the retro-analysis were also used to derive the time variation of the creep coefficient for the girder concrete, $\varphi(t, t_0)$, by fitting the result plotted in Fig. 5 to the equation:

$$\varphi(t, t_0) = C_1 \cdot \varphi_0 \cdot [\beta_c(t, t_0)]^{C_2} \quad (5)$$

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. These parameters were quantified according to the EC2. The additional parameters, C_1 and C_2 were introduced in equation (5) so that the creep model can be scaled and adjusted to the experimental results (Santos 2001).

The creep of the slab concrete was quantified according to the EC2 provisions.

Delayed deformation of the diaphragms

The evolution of shrinkage depends on the cross-sectional shape. Some shrinkage models express this dependence through the notional size, h_0 , which is defined as twice the ratio between the cross-sectional area and the perimeter exposed to drying. The notional size of the diaphragm cross section (which took the value 791 mm, based on the geometrical characteristics of the diaphragm) is significantly higher than that of the shrinkage prisms (150 mm). Consequently, shrinkage will develop slowly in the diaphragm. According to the EC2, the shrinkage of a cross section whose notional size equals 791 mm can be obtained from that of a 150 mm-notional-size cross section by multiplying the parameters S_1 and S_2 (in equation (4)) by 12.1 and 0.757 respectively. This was the strategy employed in this work to derive the shrinkage curve of the diaphragm cross section from that of the shrinkage prisms.

Besides the notional-size coefficient, creep in the diaphragm was computed using the same parameters that were employed for the slab.

Results

Comparison between numerical and experimental results

Observed and calculated results are depicted in Figs. 6 and 7 for a mid-span cross section and a cross section above the supports respectively. The observed values are represented by symbols

which identify the location of each point whereas the calculated values are depicted by lines. When more than one measurement was taken at the same cross-section depth, that location is identified with the same symbol and the graph depicts the average result at that depth. The experimental results were taken at 6 a.m. for the reason that small temperature differences occur at this time.

Fig. 6 shows the strain evolution for one of the monitored mid-span cross sections. Given that a retro-analysis procedure was employed to derive the creep and shrinkage of this concrete in the first 40 days after casting, there is a very good agreement between the experimental and the numerical results during this period of time. Nevertheless, a good agreement is also observed after this period. Construction events such as the erection of the precast planks and the casting of the slab cause important strain variations, which are clearly reproduced both in the experimental and in the numerical results. Accordingly, considerable stress changes take place at these stages. Important time variation of strains is also observed: in the case of the girder, the most relevant time-dependent strains occur in the first weeks after casting; as for the slab, its strain evolution is mainly governed by its shrinkage.

Fig. 7 presents the observed and calculated results for one of the cross sections above the pier axis. For a better understanding of the consequences of shrinkage, the observed results are compared with two calculation scenarios: one considering the diaphragm shrinkage as described before (base calculation); other considering that the shrinkage development in the diaphragm is equal to the one adopted for the deck slab. By observing Fig. 7 it can be concluded that the aforementioned scenarios represent an envelope of the long-term development of the strains in this cross section. The diaphragm shrinkage is, in fact, less than the one in the slab. However, the procedure adopted to express the dependency between the shrinkage and the notional size of the diaphragm cross section leads to an underestimation of the diaphragm shrinkage. Other researchers (Santos 2007) have also concluded that the EC2 fails to predict the relation between the shrinkage development and the notional size of the cross section. Fortunately, it was

concluded that even though the two scenarios lead to significantly different strain evolutions, those scenarios generate almost coincident results in terms of stresses (either in the mid-span or in the diaphragm cross sections) and restraint bending moments. In this way, the incertitude involved in the prediction of the diaphragm shrinkage does not affect the most relevant results in terms of practical design: the stresses and the internal forces.

Cracking was only observed in the piers which are closer to the expansion joints, as a consequence of the longitudinal displacement imposed to the top of the piers, caused by the long-term contraction of the deck. However, parametric analyses have shown that cracking of the piers does not significantly affect the results in the monitored cross sections, due to the high lateral flexibility of the piers.

The closest prediction of strains in the mid-span cross sections was obtained in the first span, alignment B (Fig. 6), since the retro-analysis was based on the monitoring of this girder in the first 40 days after casting. Nevertheless, the variability of the observed strains in the monitored cross sections was not significant, as the conclusions presented before are also applicable to the remaining cross sections.

Calculations based on the ACI209 and the EC2 creep and shrinkage models

In the previous section, it was concluded that the employed analysis strategy has led to a good agreement with the experimental results. However, the analysis was based on data which is often not available at the design stage. At that time, only limited information is available concerning the material properties and the construction sequence. Moreover, the concrete delayed deformation is estimated according to the provisions of design codes. In this section, the results obtained through the rigorous numerical analysis presented in the previous section (which are herein labeled with “base calculation”) are compared against the results of calculations based on the data that is usually available at the design stage, so that the errors due to simplified design assumptions can be ascertained. The following simplified calculations are considered.

- Two different analyses wherein the sequence of construction is the real one but the quantification of creep and shrinkage is based on the information that is usually available at the design stage. The EC2 was followed in one of the calculations and the ACI Committee 209 (1992) was considered in the other. Table 2 shows the data used in the calculations and the shrinkage predictions according to these models are shown in Figs. 3 and 4. Note that correction factors for concrete composition were not considered in the ACI creep and shrinkage models because concrete mix is usually unknown at the design stage and these correction factors are not relevant in normal cases.
- Then, two additional analyses were made in which a simplified construction sequencing is considered. In these analyses, it is considered that all the girders were cast at the same time, all the diaphragms and continuity strips of the slab were cast simultaneously at a later stage and also the mid-span portions of the slab were cast all together later on. These calculations are identified with $t_{cont}=15\text{days}$ and $t_{cont}=90\text{days}$, which means that the period of time between the pretensioning of the girders and the casting of the continuity diaphragms is 15 and 90 days, respectively. Both analyses were carried out considering the material properties as defined in the EC2.

Fig. 3 shows that both the ACI209 and the EC2 overestimate the slab shrinkage, the largest differences being detected in the case of the ACI209. On the other hand, these codes give a good prediction of the girder shrinkage after the initial four months (Fig. 4), even though important differences are found in the initial months. These results are consistent with the commonly acknowledged fact that the dispersion of shrinkage values for short-term results is much higher than for long-term ones. Furthermore, differences between observed values and code predictions were to be expected since the employed concretes contain superplasticizers and pozzolanic materials, whose effect can not be accurately reproduced by code formulae, due to the great variety of additives and different combinations used (Bazant 2001).

Given the inaccuracies in girder shrinkage predictions, it is not surprising that calculations based on the EC2 and ACI209 lead to strain evolutions different from the observed values in the initial months after casting. However, from a practical point of view, the designer is usually more interested in obtaining accurate predictions of stresses and restraint bending moments rather than in predicting the strain variation. And fortunately, the simplified calculations provide good estimates of the applied stresses and forces, as will be shown bellow. Note that it is herein assumed that the “base calculation” provides realistic results in terms of stresses and internal forces as it leads to a good agreement with the observed strains and realistic material properties were employed in that calculation.

Fig. 8 presents the evolution of the restraint bending moment in the section above the first continuity support. This graph presents exclusively the bending moments due to the concrete delayed deformation, that is, it does not include the sudden variation of bending moment due to the applied loading. It can be seen in the figure that the calculation based on the ACI209 predicts a faster development of the hogging bending moment in the first weeks after casting the deck slab, which is coherent with the fact that the ACI209 predicts higher slab shrinkage in this period, see Fig. 3. This is because a negative restraint moment develops when the slab concrete tends to shrink more than the girder.

Fig. 8 also shows that adopting a simplified sequence of construction leads to approximate results if the period of time between pretensioning and slab casting takes realistic values. The actual time intervals were 113 days and 80 days for the first and the second span, respectively. This justifies the fact that simplified analyses with $t_{cont}=90\text{days}$ lead to a close agreement with the “base calculation”, whereas important errors occur if $t_{cont}=15\text{days}$.

Fig. 8 also reveals that, at the end of the 900-day period of time, the results of the simplified calculations (apart from the one with $t_{cont}=15\text{days}$) do not differ significantly from the realistic results of the “base calculation”. Similar conclusions can be taken by analyzing the stress results in Fig. 9. This figure presents the stresses, at the end of the observation period, for

the two critical sections. When the results of the simplified calculations are compared with the “base calculation”, it becomes evident that the most significant differences occur on the top of the precast girder, in the mid-span cross section. This is a consequence of the fact that this is the location where, in this type of composite cross sections, higher stresses are caused by the differential shrinkage between the slab and the girder. In this way, this point is the most sensible to variations in the long-term behavior of the slab or the girder. However, the differences found at this location are not relevant because this point usually does not govern the design. The differences (between the results of the simplified analyses and the reference results of the “base calculation”) found in the remaining locations focused in Fig. 9 are negligible from a practical point of view, except when $t_{cont}=15$ days, which resembles the conclusions taken about the restraint bending moment.

If the aforementioned results are combined with the envelope of live loads, the structural safety can be evaluated, both in service and in ultimate limit state conditions. It was shown that calculations based on the ACI209 provide higher hogging moments. Therefore, this code leads to conservative results in the cross-sections close to the supports. But it is important to note that, if a given calculation provides higher hogging moments, then less conservative bending moments are obtained at mid-span. Therefore, time dependent calculations must provide good predictions of the actual bending moments. In this case, it was found that adequate safety exists for all the calculation scenarios.

As regards the ultimate limit state, the design value of the bending moment above P1 equals -30940 kNm (at the end of the 900-day period), which makes the differences between the different scenarios, in Fig. 8, less significant (difference of 835 kNm between calculations based on ACI209 and EC2). Moreover, the available ductility makes the discrepancies between the different calculation scenarios even less important, since a redistribution of bending moments at ultimate limit state is possible.

As regards service conditions, cracking limit state is also satisfied since the maximum tensile stress is approximately equal to the concrete tensile strength when the structure is subjected to the permanent loads and the characteristic value of live loads. This occurred for all the different scenarios, even though the ACI209 proved to be the most conservative one, as it led to the highest tensile stresses in the slab close to the supports (which was found to be the most critical location). It is worth mentioning that the largest tensile stresses do not occur inside the diaphragm. Instead, it occurs in the slab immediately after the diaphragm, because of self-equilibrated stresses due to the differential shrinkage between slab and girder. This phenomenon gives rise to non negligible tensile stresses which are added to the effects of the hogging moments.

Conclusions

- A numerical methodology for prediction of time-dependent effects in precast continuous bridges was validated through the comparison with experimental results observed in a real structure. The employed analysis strategy gives a good prediction of the structural response if the material properties are accurately modeled and the actual sequence of construction is considered.
- Even though the design codes do not accurately predict the actual development of the concrete delayed deformations, calculations carried out according to these codes may lead to a good estimate of the final restraint moments if realistic values for the girder age at the time that the continuity is established are adopted in the analysis.
- In this structure, the difference between actual shrinkage values and code predictions decrease as the observation period increases. This is consistent with the commonly acknowledged fact that shrinkage variability is higher for short-term tests.

- Seasonal shrinkage variations lead to a slight seasonal variation of the structural response. This effect needs not to be taken into account in design calculations as it does not originate relevant stresses or deformations.
- The actual time variation of strains in the diaphragms was not precisely predicted. But it was shown that the uncertainty involved in the prediction of the diaphragm strains does not affect the most relevant results in terms of practical design: the stresses and the internal forces.
- Self-equilibrated stresses due to differential shrinkage must not be disregarded, as they give rise to tensile stresses which have an unfavorable effect in cross sections close to the supports.
- The experimental results observed in this precast continuous deck validated the numerical models, which in turn showed that adequate safety exists, both in service and at ultimate limit state.

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References

- ACI Committee 209 (1992). "Prediction of creep, shrinkage, and temperature effects in concrete structures (ACI 209R-92)." American Concrete Institute.
- Barr, P. J., Kukay, B. M. and Halling, M. W. (2008). "Comparison of prestress losses for a prestress concrete bridge made with high-performance concrete." *Journal of Bridge Engineering*, 13(5), 468-475.
- Bazant, Z. P. (2001). "Prediction of concrete creep and shrinkage: past, present and future." *Nuclear Engineering and design*, 203(1), 27-38.
- Borst, R. (1991). "Computational methods in non-linear solid mechanics. Part 2: physical non-linearity." Delft University of Technology, Delft, The Netherlands.
- CEN (2004). "EN 1992-1-1 - Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings." Brussels.
- Cruz, P. J. S., Wisniewski, D. F. (2004). "Ave River Bridge - A Major Precast Prestressed Concrete U-Girder Bridge in Portugal." *PCI Journal*, Jul-Aug, 72-85.
- Figueiras, J. A. (1983). "Ultimate load analysis of anisotropic and reinforced concrete plates and shells." PhD Thesis, Department of Civil Eng., Univ. College of Swansea.
- Freyermuth, A. H. (1969). "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders." *PCI Journal*, 14(2), 14-39.
- Ghali, A., Favre, R. and Elbadry, M. (2002). "Concrete Structures: Stresses and Deformations." 3rd. Ed. Spon Press.
- Hastak, M., Mirmiran, A., Miller, R., Shah, R. and Castrodale, R. (2003). "State of practice for positive moment connections in prestressed concrete girders made continuous." *Journal of Bridge Engineering*, 8(5), 267-272.
- Kwak, H. G. and Seo, Y. J. (2000). "Long-term behavior of composite girder bridges." *Computers and Structures*, 74(5), 583-599.

- Ma, Z., Huo, X., Tadros, M. K. and Baishya, M. (1998). "Restraint Moments in Precast/Prestressed Concrete Continuous Bridges." *PCI Journal*, Nov-Dec, 40-57.
- Marí, A. R. (2000). "Numerical simulation of the segmental construction of three dimensional concrete frames." *Engineering Structures*, 22(6), 585-596.
- Mattock, A. H. (1961). "Precast-prestressed concrete bridges 5: Creep and shrinkage studies." *J. PCA Res. Devel. Lab.*, 3(2), 32-66.
- McDonagh, M. D., Hinkley, K. B. (2003). "Resolving Restraint Moments and Designing for Continuity in Precast Prestressed Concrete Girder Bridges." *PCI Journal*, Jul-Aug, 2-17.
- Miller, R., Castrodale, R., Mirmiran, A. and Hastak, M. (2004). "NCHRP Report 519 - Connection of Simple-Span Precast Concrete Girders for Continuity." *Transportation Research Board*, Washington, D.C.
- Mirmiran, A., Kulkarni, S., Castrodale, R., Miller, R., Hastak, M. (2001). "Nonlinear Continuity Analysis of Precast, Prestressed Concrete Girders with Cast-in-Place Decks and Diaphragms." *PCI Journal*, 46(5), 60-80.
- Newhouse, C. D., Roberts-Wollmann, C. L., Cousins, T. E. and Davis, R. T. (2008). "Modeling early-age bridge restraint moments: Creep, shrinkage, and temperature effects." *Journal of Bridge Engineering*, 13(5), 431-438.
- Osterle, R. G., Gilkin, J. D. and Larson, S. C. (1989). "NCHRP Report 322 - Design of precast-prestressed bridge girders made continuous." *Transportation Research Board*, Washington, D.C.
- Peterman, R. J., Ramirez, J. A. (1998). "Restraint Moments in Bridges with Full-Span Prestressed Concrete Form Panels." *PCI Journal*, Jan-Feb, 54-73.
- Santos, L. M. (2001). "Observação e análise do comportamento diferido de pontes de betão." PhD Thesis, Instituto Superior Técnico da Univ. Técnica de Lisboa (in Portuguese).
- Santos, T. O. (2007). "Retração do betão em pontes. Observação e análise." PhD Thesis, Faculdade de Ciências e Tecnologia da Univ. Nova de Lisboa (in Portuguese).
- Sousa, C. and Neves, A. (2009). "DIANA user-supplied subroutine for concrete creep modelling." *LABEST*, Faculty of Eng. of the Univ. of Porto.

Sousa, C., Sousa, H., Neves, A. and Figueiras, J. (2009). “Finite-element analysis of the long-term behavior of the precast access viaduct of the Leziria Bridge.” LABEST, Faculty of Eng. of the Univ. of Porto.

Valdés, M. (1997). “Behavior during construction and under permanent loads of continuous precast concrete bridges.” PhD Thesis, E. T. S. Ingenieros de Caminos, Univ. Politécnica de Catalunya (in Spanish).

Witte, F. C. (2005). “DIANA user's manual, Release 9.” TNO DIANA, BV, Delft, The Netherlands.

Witte, F. C. and Kikstra, W. P. (2005). “DIANA user's manual - Material Library.” TNO DIANA, BV, Delft, The Netherlands.

Table 1. Concrete properties based on cube compression tests

	$f_{cm,cube}$ (MPa)	s
Piles	60.47	0.350
Piers (above the ground)	46.65	0.250
Precast beams ^(a)	67.80	0.186
CIP slab (above the piers) ^(a)	67.57	0.170
CIP slab (mid span) ^(a)	68.78	0.196

Note: ^(a) Average of the values obtained for the first and the second spans (where monitoring was carried out)

Table 2. Data for creep and shrinkage calculations based on the EC2 and the ACI209

	Girders	CIP slab	Diaphragms
f_{cm} (MPa)	48	38	38
h_0 (mm)	^(a)	250	791
Relative humidity (%)	80	80	80
Curing time (days)	^(b)	1	1
Cement type	Rapid-hardening		
Initial curing	steam	moist	moist

Notes:

^(a) h_0 takes the values 182mm or 202mm depending on the web thickness, which equals 150mm (close to the supports) or 200mm (in the mid-span region)

^(b) drying begins at the time of prestress release

Fig. 1. Longitudinal section of the structure and monitored cross sections

Fig. 2. Cross section and layout of vibrating-wire strain gauges: a) mid-span; b) above the pier axis

Fig. 3. Shrinkage of the slab concrete

Fig. 4. Shrinkage of the girder concrete

Fig. 5. Observed strains and results of the retro-analysis

Fig. 6. Concrete deformation in the mid-span cross section (first span; alignment B)

Fig. 7. Concrete deformation in the cross section above P1 (alignment A)

Fig. 8. Restraint bending moment at the cross section above P1

Fig. 9. Stresses at the end of the observation period: a) middle section of the first span; b) section above P1