Role of uncertainties on time dependent behavior of prestressed and cable stayed concrete bridges

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ABSTRACT: The behavior of pre-stressed and of cable stayed concrete bridges may be strongly affected by the time dependent response due to the creep and shrinkage of concrete. The assessment of safety and serviceability performance of these structures during their whole service life requires a reliable estimation of the influence of these phenomena.

The conventional design of pre-stress losses and, in general, of the time dependent behavior of non homogeneous structures, like cable stayed bridges, uses tools of analysis which involve many uncertain quantities, such as environmental factors, the material characteristics and the intensities of the tension in cables. Also construction imperfections may strongly influence the final structural response.

After some brief recalls regarding the time dependent analysis in presence of creep and shrinkage, this paper studies, through a probabilistic approach, the effects due to a large variations of the basic variables.
Two cases are studied: the first one is less sensitive to uncertain data, while the second is strongly affected by them.

In the case of a cable stayed bridge, made of a concrete deck, subjected to creep effects and suspended to a set of pretensioned cables, the role of uncertainties in the pretensioning forces, in the relative humidity and in the concrete strength do not influence the time dependent behavior sensibly. The standard deviation with respect to the mean tension in the cables is relatively small and, most of all, it progressively reduces. Hence the system seems to be self-stabilizing over time.

On the contrary, in the case of a prestressed cantilever beam, the effects of uncertainties in the pretensioning forces and in the concrete strength cause a significant variance of the tip deflections. Both deflections and their variance increase over time and, speaking about bridges, they may strongly modify the vertical attitude of the structure. These effects are emphasized when the prestressing is applied a few days after curing.
Such results outline the limits of the traditional deterministic analyses and suggest the need of further studies on this topic.

1 INTRODUCTION

In many cases the time dependent behaviour of reinforced and prestressed concrete structures has little relevance, with respect to both the serviceability and the ultimate states.

In other cases, the structural behavior may be strongly affected by the time dependent effects due to creep and shrinkage of concrete, like in the case of pre-stressed and of cable stayed suspended concrete bridges (Martinez y Cabrera 1997, Smerda et al. 2006).

The usual design process deals with these structures through time dependent deterministic analyses and assumes geometry, material characteristics, tensioning forces and all the other factors involved in the structural behavior as known and certain data. These data are usually derived from national standards, from technical reports or from the outcomes of preliminary acceptance tests. Sometimes they are based on the designer’s previous experiences.

However, the same experiences showed us that, even after accurate design assessments, some unsatisfactory service behavior can arise, due to some level of uncertainty in the reference data (Sgambi 2005).

The main sources of uncertainty derives from the material characteristics and from the construction process. The actual material characteristics, in particular the concrete strength, and the environmental parameters, like temperature and relative humidity (RH), influence the creep phenomena and the corresponding creep model. The technological factors may influence the actual effectiveness of the pretensioning/prestressing actions.

After some brief recall regarding the time dependent analysis in presence of creep and shrinkage, this paper studies, through a probabilistic approach, the effects due to the large variations of the basic variables.

Two significant cases are studied, the first concerning a cable stayed suspended concrete deck, and the second one concerning a prestressed cantilever concrete beam. The first one resulted less sensitive to uncertain data, while the second resulted strongly affected by them (Camossi 2009).

2 SOME RECALLS OF VISCOELASTICITY

2.1 *Stress-strain relationships*

For a sustained stress applied at time t_0, the total strain at time t is

$$\begin{aligned}\varepsilon_c(t,t_0) &= \sigma_c(t_o)/E_c(t_0)\cdot\left[1+\varphi(t,t_0)\right]= \\ &=\sigma_c(t_0)\cdot f(t,t_0)\end{aligned} \tag{1}$$

where $\varphi(t,t_0)$ is the dimensionless creep coefficient, function of the age of the concrete t_0 at loading and of the age t when the strain is measured (Neville et al. 1983, Ghali et al. 2002). Assuming that the principle of superposition holds, the strain corresponding to a stress history composed of two basic stress histories equals the sum of the strains due to each of these and, in general, if the stress intensity varies with time, the total strain of the concrete due to the applied stress is given by:

$$\begin{aligned}\varepsilon_c(t,t_0) &= \frac{\sigma_c(t_o)}{E_c(t_0)}\cdot\left[1+\varphi(t,t_0)\right]+ \\ &+\int_{\sigma_c(t_0)}^{\sigma_c(t)}\frac{\left[1+\varphi(t,\tau)\right]}{E_c(\tau)}\cdot d\sigma_c(\tau)\end{aligned} \tag{2}$$

where the integral must be understood as a Stieltjes integral.

An efficient way to reduce the Equation 2 to an algebraic form, suitable for practical applications, is the Age Adjusted Effective Modulus Method (A.A.E.M.M.), (Bazant 1972, CEB-FIP 1984), through which the integral at the second member is expressed as follows:

$$\begin{aligned}&\int_{\sigma_c(t_0)}^{\sigma_c(t)}\frac{\left[1+\varphi(t,\tau)\right]}{E_c(\tau)}\cdot d\sigma_c(\tau)= \\ &=\Delta\sigma_c(t)\cdot\frac{\left[1+\chi(t,t_0)\cdot\varphi(t,t_0)\right]}{E_c(t_o)}= \\ &=\Delta\sigma_c(t)\cdot\frac{\left[1+\chi\cdot\varphi(t,t_0)\right]}{E_c(t_0)}\end{aligned} \tag{3}$$

where $\Delta\sigma_c(t)=\sigma_c(t)-\sigma_c(t_0)$. The aging coefficient $\chi=\chi(t,t_0)$ is given by

$$\begin{aligned}\chi(t,t_0) &= \frac{E_c(t_0)}{\varphi(t,t_0)}\cdot\int_{t_0}^{t}\frac{\left[1+\varphi(t,\tau)\right]}{E_c(\tau)}\cdot\frac{d\xi}{d\tau}\cdot d\tau - \\ &-\frac{1}{\varphi(t,t_0)}\end{aligned} \tag{4}$$

with $\sigma_c(\tau)=\sigma_c(t_0)+\xi(\tau)\cdot\Delta\sigma_c(t)$ and $\xi(\tau)$ a dimensionless time function ($0\le\xi(\tau)\le 1$, for $t_0\le\tau\le t$) defining the shape of the stress-time curve.

The exact evaluation of $\chi=\chi(t,t_0)$ depends on the stress or strain variation with time. Bazant (Bazant 1972) pointed out that the method is theoretically exact for any problem in which the strain varies proportionally with the creep coefficient. In these cases, calling $r(t,t_0)$ the relaxation function, the aging coefficient χ is given by

$$\chi(t,t_0)=1/\left[1-r(t,t_0)/E_c(t_0)\right]-1/\varphi(t,t_0) \tag{5}$$

In most practical creep problems the strain variation can be assumed not too far from this assumption and so, many technical problems can be reduced to the above category (Mola et al. 1987, Gilbert 1988, Sgambi et al. 2003). Through Equation 3, the Equation 2 becomes:

$$\begin{aligned}\varepsilon_c(t,t_0) &= \sigma_c(t,t_o)\cdot\frac{\left[1+\varphi(t,t_0)\right]}{E_c(t_0)}+ \\ &+\left[\sigma_c(t)-\sigma_c(t_o)\right]\cdot\frac{\left[1+\chi\cdot\varphi(t,t_0)\right]}{E_c(t_0)}\end{aligned} \tag{6}$$

which, by putting

$$\chi\cdot\varphi\equiv\chi\cdot\varphi(t,t_0)\equiv\chi(t,t_0)\cdot\varphi(t,t_0) \tag{7}$$

can be rearranged as follows:

$$\varepsilon_c(t,t_0)=\frac{\sigma_c(t)\cdot\left[1+\chi\cdot\varphi\right]}{E_c(t_0)}+\frac{\sigma_c(t_o)}{E_c(t_0)}\cdot\varphi\cdot(1-\chi) \tag{8}$$

Introducing the Age Adjusted Elasticity Modulus $\overline{E}_c(t)=E_c(t_0)/\left[1+\chi\cdot\varphi(t,t_0)\right]$ and regarding $\overline{\varepsilon}_c(t)=\left[\sigma_c(t_0)/E_c(t_0)\right]\cdot\varphi(t,t_0)\cdot(1-\chi)$ as an imposed deformation, the Equation 8 can be expressed in the following pseudo-elastic form:

$$\varepsilon_c(t,t_0)=\sigma_c(t)/\overline{E}_c(t)+\overline{\varepsilon}(t) \tag{9}$$

which will be used in the following for the structural analysis.

2.2 *Structural* analysis

The structural analysis is focused on frame structures and is carried out through the Displacement Method. With reference to a given time *t*, each step of analysis involves two steps:

- Step 1: an elastic analysis at time $t = t_0$;
- Step 2: a viscoelastic analysis at time t, which cumulates the effects due to creep. Such effects are computed by solving the superposition integral through the A.A.E.M. Method.

At step 1, at time $t = t_0$, each element *e* of the structure has the elasticity modulus $E = E_c(t_0)_e$ and is subjected to the given applied loads and to the imposed deformations corresponding to the prestressing/pretensioning transmission. The outcomes of this step of analysis are the vectors of the nodal displacements $\underline{s}(t_0)_e$ and of the nodal forces $\underline{f}(t_0)_e$.

At Step 2, at time t, the structure has the age adjusted elasticity modulus $\overline{E}_c(t,t_0)$. The equivalent nodal forces applied at time t_0 are assumed as constant and do not concern this phase of analysis. The active forces at time t are those which give nodal displacements equal to those due to creep. The equivalent nodal force vector for the element *e* is:

$$\underline{f}(t,t_0)_e = \frac{\overline{E}_c(t,t_0)_e}{E_c(t_0)_e} \cdot \varphi(t,t_0)_e \cdot f_S(t_0)_e \qquad (10)$$

where $\underline{f}_S(t_0)_e$ is the product of the element stiffness matrix $\underline{\underline{K}}(t_0)_e = E_c(t_0)_e \cdot \underline{\underline{\hat{K}}}(t_0)_e$ and of the nodal displacement vector $\underline{s}(t_0)_e$:

$$\begin{aligned} \underline{f}_S(t_0)_e &= \underline{\underline{K}}(t_0)_e \cdot s(t,t_0)_e \\ &= E_c(t_0)_e \cdot \underline{\underline{\hat{K}}}(t_0)_e \cdot s(t_0)_e \cdot \varphi(t,t_0)_e \end{aligned} \qquad (11)$$

The vector of the equivalent nodal forces to be applied at the element *e* at time t is:

$$\underline{f}_{eq}(t,t_0)_e = E_c(t_0)_e \cdot \underline{\underline{\hat{K}}}(t_0)_e \cdot \underline{s}(t_0)_e \cdot \varphi(t,t_0)_e \qquad (12)$$

3 THE ROLE OF THE UNCERTAINTIES. A SENSITIVITY ANALYSIS

As mentioned before, such type of analyses involves many uncertain quantities. We want to assess the role of these uncertainties on the structural behavior over time.

Different set of uncertain parameters are considered (environmental factors, material characteristics, tension in the cables). We assume these parameters as random variables, characterized by a mean value and by a standard deviation and modelled through Gaussian distributions (Elishakoff, I. 1983).

We want to assess the sensitivity of structural systems with respect to a certain levels of the variance of the initial data. Such a sensitivity should be measured on quantities concerning the safety and/or the serviceability of the structure. For instance, in a cable stayed bridge these quantities should be the tension in the cables. In a cantilever system should be a displacement of a particular point, assumed as indicator of an excessive deformed shape, which may limit the serviceability of the structure.

In particular we want to ascertain if the load effects increase with time and if their variance increases too, as shown in Figure 1.A.

But, due to some internal redistribution mechanism, we can not exclude that the load effects remain quite stable over time. For the same mechanism a reduction of the variance is possible (Figure 1.B).

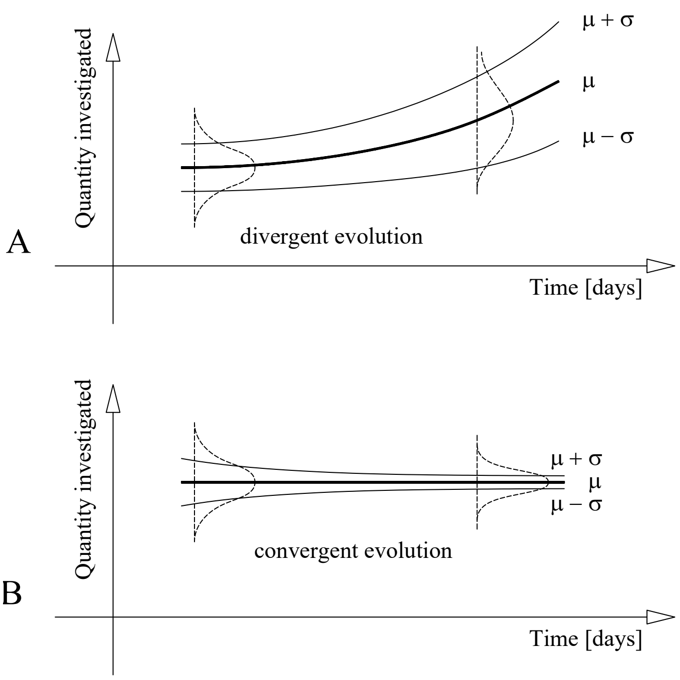

Figure 1. Evolution of the structural behavior over time: divergent (1.A) and convergent (1.B).

The sensitivity analysis is carried out through Monte Carlo simulations, by repeating the analyses on a certain number of samples. More details are given in the following, dealing the case studied.

4 ROLE OF THE UNCERTAINTIES IN TIME DEPENDENT BEHAVIOUR OF A CABLE STAYED BRIDGE

We consider a cable stayed bridge having the geometry and the sections shown in Figure 2 and suspended on a set of (2+2) stays (Martinez y Cabrera 1997). We suppose that the stays were tensioned after 28 days from casting.

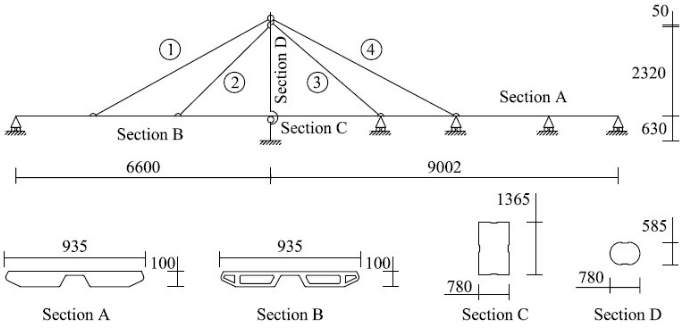

Figure 2. Geometrical characteristics of the bridge (Martinez y Cabrera et al. 1997).

Two analyses are performed. The first one (case A) intends to assess the role of the uncertainties in the initial pretensioning of the stays on the time dependent behaviour of the bridge. The second analysis (case B) would assess the importance of the uncertainties due to environmental and material characteristics.

4.1 *Case A) Uncertainty about the initial pretensioning values*

The pretension intensities are modelled through Gaussian distributions, having the mean and the standard deviation reported in Table 1. The vertical components of the mean values correspond to the condition of rigid support. As known, for these values of pretensioning, the forces in the stays remain constant in time (Martinez et al. 1997).

Table 1. Cable pretensioning forces. Probability distributions and their parameters (mean value μ and standard deviation σ).

Element	μ (mean)	σ (s.d.)
Cable 1	5260 kN	10%
Cable 2	3373 kN	10%
Cable 3	3765 kN	10%
Cable 4	5437 kN	10%

The structural analyses were carried out by means of the A.A.E.M. Method. As measure times 28, 29, 31, 38, 344, 1000 and 10000 days from casting were assumed. The probabilistic analysis was performed by a Monte Carlo simulation, based on 3000 samples.

Figure 3 shows how the tension in stays No.1 and 2 over time. The effects due, respectively, to the self-weight, to the pretensioning and to the sum of these two contributions are separately presented. The time dependent analysis for the self-weight alone is deterministic, being the geometry not affected by uncertainties. The three lines, associated to the pretensioning and to total results, represent the mean μ (thick line) and the standard deviation σ from the mean μ (thin lines).

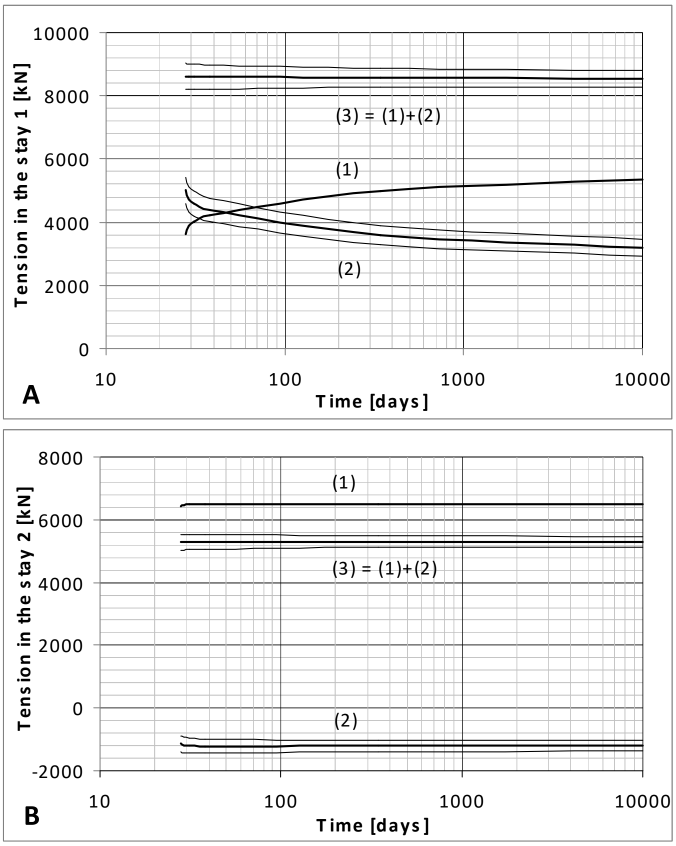

Figure 3. Effects of uncertainties in the initial pretensioning of the stays. Tension variation in stays No.1 (3.A) and 2 (3.B) over time. (1) self-weight only; (2) pretensioning only; (3)=(1)+(2): mean value μ (thick line) and standard deviation σ from the mean μ (thin lines).

According to these curves, we can deduce that, even in presence of significant deviations from the nominal values of the initial pretensioning, the changes over time remain in a narrow band and that these trend tends, with time, to converge towards the mean value.

4.2 *Case B) Uncertainty due to environmental exposition and material characteristics*

This second analysis evaluates the influence on the final tension of the cables with uncertainties related to one environmental factor, the relative humidity *RH*, to the material characteristics, to the concrete strength f_{ck}, and to the time of pretensioning t_0. Table 2 lists the mean value and the standard deviation of these quantities.

Table 2. Influence of the relative humidity, concrete strength, and time of pretensioning. Probability distributions and their parameters (mean value μ and standard deviation σ).

Element or parameter	μ (mean)	σ (s.d.)
Cable 1	5260 kN	10%
Cable 2	3373 kN	10%
Cable 3	3765 kN	10%
Cable 4	5437 kN	10%
RH	65%	20%
f_{ck}	48.1 MPa	20%
t_0	28 gg	20%

The resulting tensioning levels are computed at 28, 29, 31, 38, 344, 1000 and 10000 days from casting.

Figures 4 shows the tension variation in stays No.1 and 2 over time. In this case, the effects of the self-weight are also uncertain, as they depend on the parameters governing the creep characteristics (*RH*, f_{ck}, t_0). According to these curves, we can deduce that, even in presence of significant deviations from the nominal values of the initial pretensioning, the changes over time remain in a narrow band. At time $t=t_0$ we obtain μ=8.618·E+3 kN, $\mu+\sigma$ =9.047·E+3 kN and $\mu-\sigma$ = 8.190·E+3 kN. Moreover we observe that this trend tends, with time, to converge towards the mean value. At time t=10000 days we obtain μ=8.545·E+3 kN, $\mu+\sigma$ =8.812·E+3 kN and $\mu-\sigma$ = 8.278·E+3 kN.

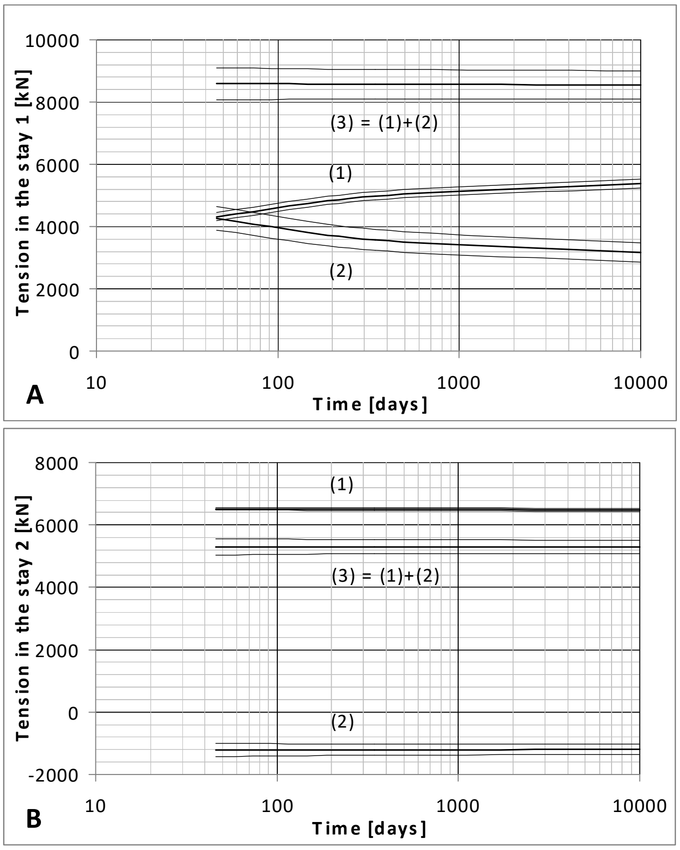

Figure 4. Influence of relative humidity, concrete strength, and time of pretensioning. Tension variation in stays No.1 (4.A) and 2 (4.B) over time. (1) self-weight only; (2) pretensioning only; (3)=(1)+(2): mean value μ (thick line) and standard deviation σ from the mean μ (thin lines).

5 ROLE OF THE UNCERTAINTIES ON THE TIME DEPENDENT BEHAVIOUR OF A PRESTRESSED CANTILEVER BRIDGE

We consider a bridge made of short cantilevers spanning 28.5 m and supporting closing suspended girders, 10.0 m long. The free length of each cantilever is 25.0 m. The length between two piers of a typical reach is 67.0 m. The transversal section is made of a double box girder 11.0 m wide. The piers are made of a couple of parallel reinforced concrete blades. Figure 5 shows a front view of a half span of the bridge and the main transversal sections. Each vertical web of the box lodges twenty five cables, made of 12∅7 mm wires. The cable layout is shown in Figure 6. Each cantilever is a statically determinate structure and it is considered as independent. In this example, we suppose the cantilever was cast on a false work and prestressed in only one stage after curing.

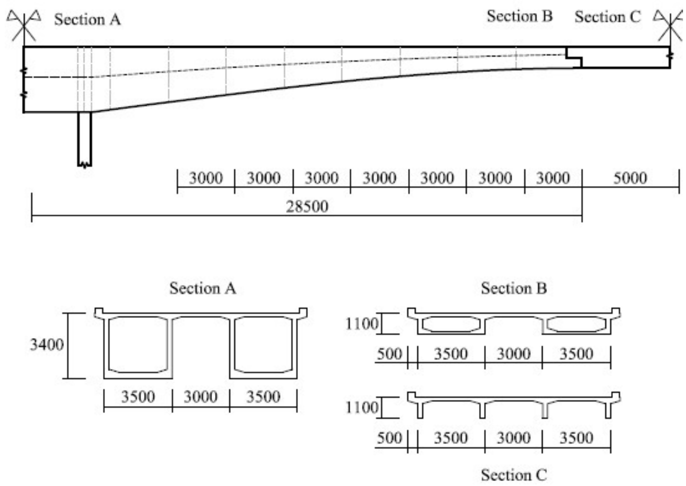

Figure 5. Front view of a half span and main sections of the bridge.

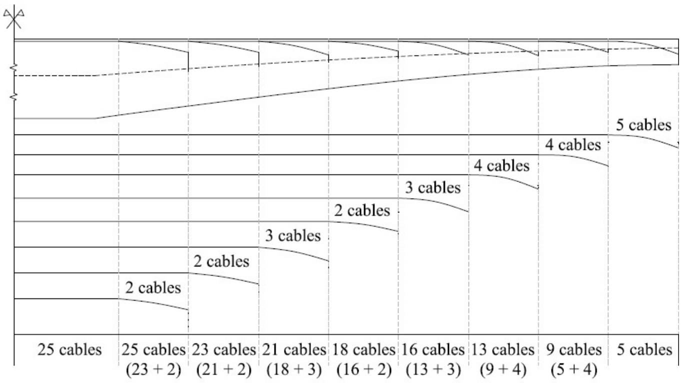

Figure 6. Tendons layout.

Through the A.A.E.M. Method we define as reference prestressing forces in the cables those which balance the long term deformations due to the dead loads. Figure 7 shows the time dependent vertical displacement at the tip of the cantilever due, respectively, to the dead load (curve 1) and to the cable prestressing (curve 2). Curve 3 shows the combination of the two previous effects, which remains constant over time.

By using Gaussian distributions, the influence of uncertainties associated to cables prestressing forces and to concrete strength f_{ck} are investigated. Table 3 reports the mean value and the standard deviation of these quantities.

The large spread assumed in the prestressing force intensities is meant to take into account the uncertainties due to the prestressing losses and to construction imperfections.

Table 3. Mean and variance of the probabilistic data assumed.

Element or parameter	μ (mean)	σ (s.d.)
Prestressing in cable i	388.6 kN	50%
f_{ck}	35 MPa	10%

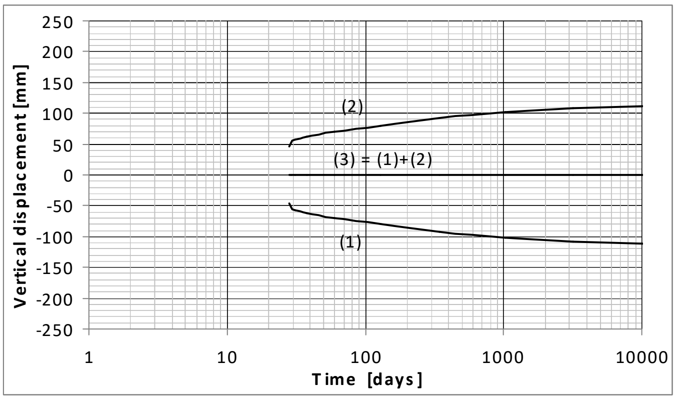

Figure 7. Vertical displacements at the tip of the cantilever beam over time. (1) dead loads only; (2) prestressing only; (3) = (1)+(2).

Two type of analyses were developed. Conventionally, in order to have a reference state, the first one assumes the prestressing transmission 28 days (t_0) after the casting of each segment. A second group of analyses assumes the prestressing transmission after 3.5 days after casting.

5.1 *Case A) prestressing at t_0 = 28 days*

The structural analyses were carried out by means of the A.A.E.M. Method. The analyses were performed at the age of 28, 29, 31, 38, 344, 1000 and 10000 days. The probabilistic analysis was performed by a Monte Carlo simulation, based on 3000 samples.

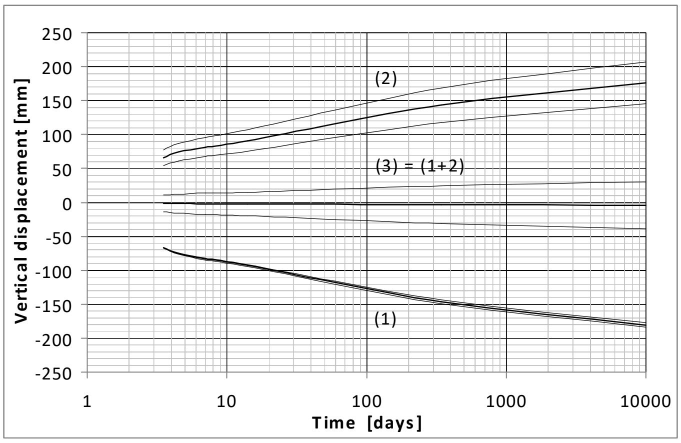

Figure 8. Vertical displacement of the tip of the cantilever over time. Influence of the cable prestressing and of the concrete strength. (1) dead loads only; (2) prestressing only; (3)=(1)+(2): mean value μ (thick line) and standard deviation σ from the mean μ (thin lines).

Figure 8 shows the vertical displacements at the tip of the cantilever plotted over time and due to the dead load, to the prestressing and to the sum of the two effects. Each set of three lines represents the mean μ (thick line) and the standard deviation σ from the mean μ (thin lines).

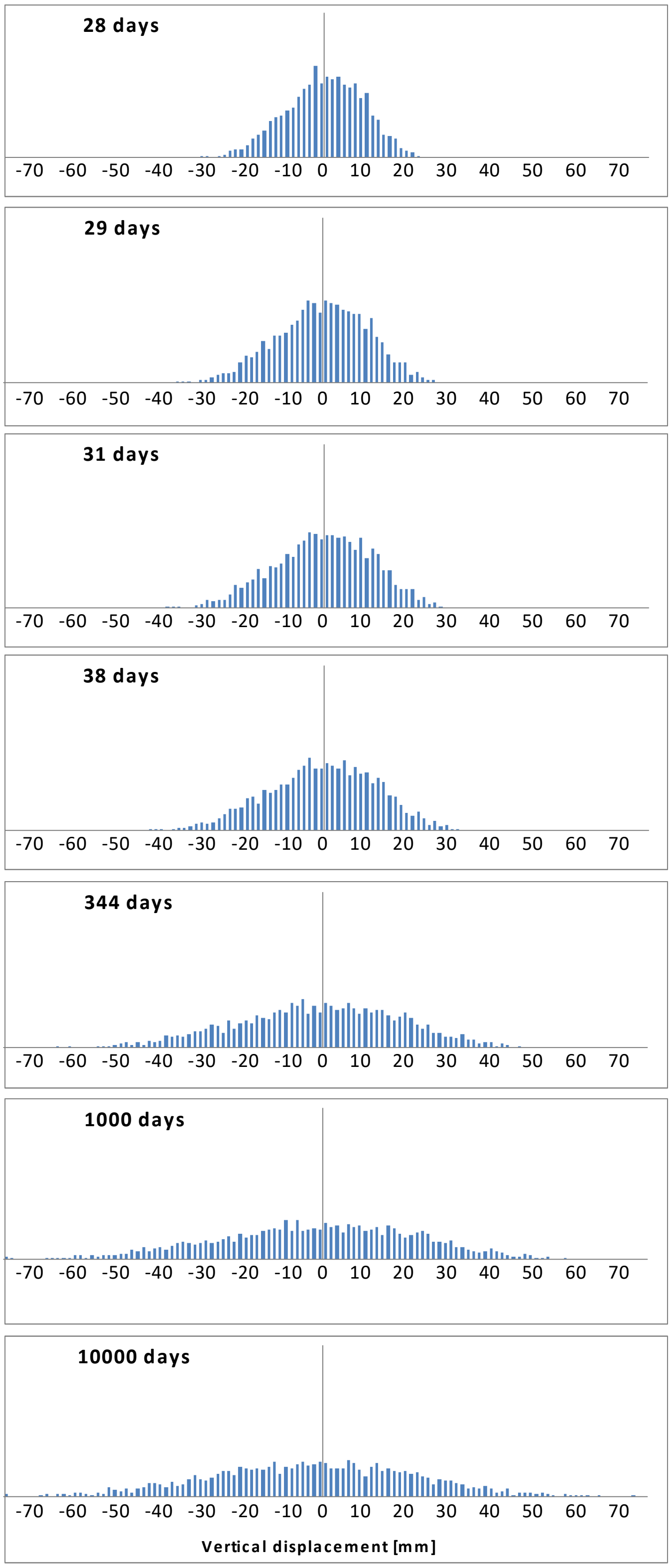

Figure 9. Prestressing time t_0 = 28 days. PDF of the vertical displacements at the tip of the cantilever for each time of analysis.

With the values assumed for the standard deviation of the prestressing forces, it is obvious that such type of uncertainty plays a leading role. However, the uncertainties in the resulting deflections increase with time, as it can be deduced from the increase of their variance, shown in Figure 9 for each time of analysis. At t = 10.000 days, the deviation from zero of the mean deflection is -3.03 mm and the difference $\mu \pm \sigma$ is (+18.91)÷(-24.98) mm.

5.2 *Case B) prestressing at t_0 = 3.5 days*

As known, the creep effects are emphasized when loads and prestressing are applied a short time after curing. By assuming a prestressing time t_0 = 3.5 days and repeating the simulation, the result of Figure 10 are obtained.

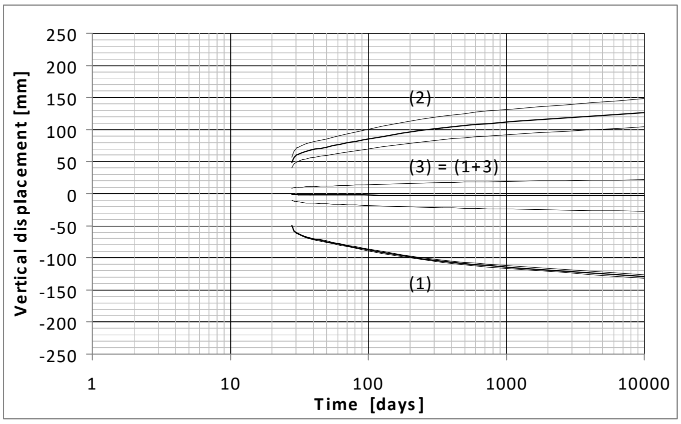

Figure 10. Prestressing time t_0 = 3.5 days. Vertical displacements at the tip of the cantilever over time. (1) dead loads only; (2) prestressing only; (3) = (1)+(2).

At t = 10000 days, the deviation from zero of the mean deflection is -4.24 mm and the difference $\mu \pm \sigma$ is (+26.45)÷(−34.93) mm, are considerably higher than in the previous case, as well as the standard deviation.

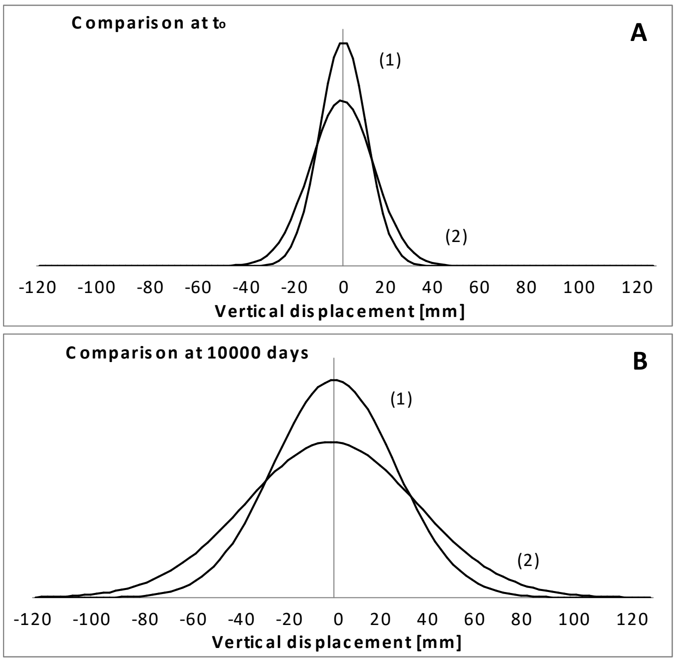

Figure 11. PDF of the vertical displacements at the tip of the cantilever at $t=t_0$ (11.A) and t=10000 days (11.B). Case (1) t_0=28 days. Case (2) t_0=3.5 days.

Figure 11 compares the PDF of the vertical displacements at the tip of the cantilever at $t=t_0$ and at t=10000 days for the cases A) and B). At t=10000 days, the standard deviation increases from σ = 21.94 mm to σ = 30.69 mm (+39.9%).

6 CONCLUSIONS

The role of uncertainties on the time dependent behavior of prestressed and cable stayed concrete beams has been investigated.

As sources of uncertainty, the pretensioning forces in the cables, the relative humidity and the concrete strength have been considered.

For a given set of data, the creep effects has been handled by means of the A.A.E.M. Method. The effects of the uncertainties have been simulated through a Monte Carlo approach.

Two topical cases have been studied: one less sensitive to uncertain data, the other one strongly affected by them.

In the case of a cable stayed bridge, made of a concrete deck, subjected to creep effects and suspended to a set of pretensioned cables, the role of uncertainties in the pretensioning forces, in the relative humidity and in the concrete strength do not influence the time dependent behavior sensibly. The standard deviation with respect to the mean tension in the cables is relatively small and, most of all, it progressively reduces. Hence the system seems to be self stabilizing over time.

On the contrary, for the case of a prestressed cantilever beam, the effects of uncertainties in the pretensioning forces and in the concrete strength cause a relevant variance of the tip deflections. Both deflections and their variance increase with time and, in the case of bridges, they may strongly modify the vertical attitude of the structure. These effects are emphasized when the prestressing is applied a few days after curing.

This contribution touches two relevant themes of the bridge design: the cable pretensioning set-up in the cable stayed bridges and the theoretical definition of the attitude of cantilever beams.

At the moment, this would be the first assessment of the sensitivity of these systems with respect to the uncertainties in the main factors which govern the structural behavior. The achieved results and, in particular, those concerning the prestressed cantilever beams, outline the limits of the traditional deterministic analyses and suggest further studies, suitable in order to deal with all the complexities of the actual characteristics of the real structures.

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