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Analysis of the variability of deflection of a prestressed composite bridge deck

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Abstract. Nearly 400 composite railway bridge decks of a new kind belonging to the trough type with U-shaped cross section have been constructed in Belgium over the last fifteen years. The construction of these bridge decks is rather complex with the preflexion of precambered steel girders, the prestressing of a concrete slab and the addition of a 2nd phase concrete. Until now, they have been designed with a classical computation method using a pseudo-elastic analysis with modular ratios. Globally, they perform according to the expectations but variability has been observed between the measured and the computed camber of these bridge decks just after the transfer of prestressing and also at long-term. A statistical analysis of the variability of the relative difference between the measured camber and the computed camber is made for a sample of 36 bridge decks using no less than 10 variables. The most significant variables to explain this variability at prestressing are the ratio between the maximum tensile stress reached in the steel girders during the preflexion and the yield strength and the type of steel girder. For the same sample, the long-term camber under permanent loading is computed by two methods and compared with measurements taken one or two years after the construction. The camber computed by the step-by-step method shows a better agreement with the measured camber than the camber computed by the classical method. The purpose of the paper is to report on the statistical analysis which was used to determine the most significant parameters to consider in the modeling in order to improve the prediction of the behaviour of these composite railway bridge decks.

Key words: camber; composite bridge; high strength concrete; hot-rolled girder; numerical modelling; prestressing; statistical analysis; welded girder.

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1. Introduction

A new kind of railway bridge deck has been developed recently in Belgium for the replacement of old steel railway bridges with moderate spans and for the construction of multi-spans viaducts for the new high speed lines. Up to now, these bridge decks have been used (single track) for simply supported spans up to 26 m. The bridge decks are prefabricated in workshops and transported by train to the construction site where they are placed on their supports by cranes. These composite steel-concrete prestressed structures belong to the trough type. Their U shaped cross section, with a breadth of 4 m, is represented in Fig. 1.

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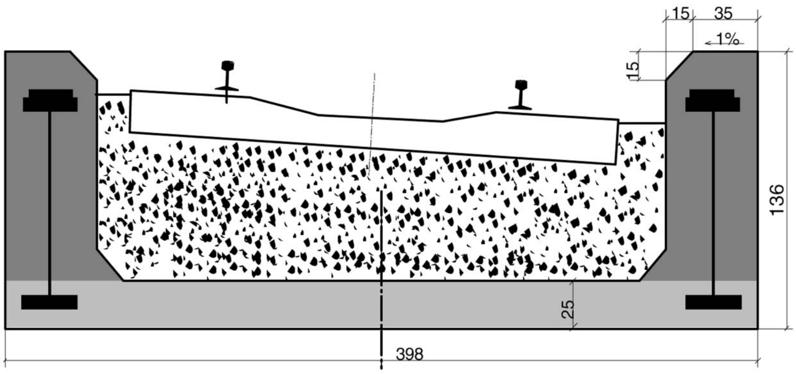

Fig. 1 Typical cross section of a through shaped composite railway prestressed bridge deck

The two high strength steel hot-rolled (HEA1000, HEB1000,…) or welded I-girders are bent at the mill or in plant to produce an initial camber (Fig. 2a). Then, the first step in the workshop is the elastification phase of the steel girders. In order to remove the residual stresses, two local loads are applied at ¼ and ¾ of the span of the steel girders, released and applied again several times until the precamber does not change any more. Then, the preflexion phase is carried out by applying the local loads again on each steel girder at ¼ and ¾ of the span in order to straighten the girders and to obtain at this stage a camber equal to zero (Fig. 2b). The stress level in the steel girders during this preflexion phase is limited to 80% of the yield strength. These two girders will be parts of the webs of the bridge. Then, the bottom slab of the deck is constructed: reinforcing bars (transversally and longitudinally) and naked tendons (grade=1840 MPa) (longitudinally) are disposed and stressed in the space that will be filled by the concrete bottom slab (slab depth: 0.25 m). The bottom slab is concreted (grade C60) some hours after the preflexion of the steel girders (Fig. 2c). The bridge decks prestressed at a very early age are heated at 45°C during the first day after casting. At 40 hours (for the decks with heat curing) or 62 hours (mainly for the non heated ones) of age, the bottom slab is prestressed by releasing the preflexion of the steel girders and by transferring the prestressing force from the tendons (Fig. 2d). On the

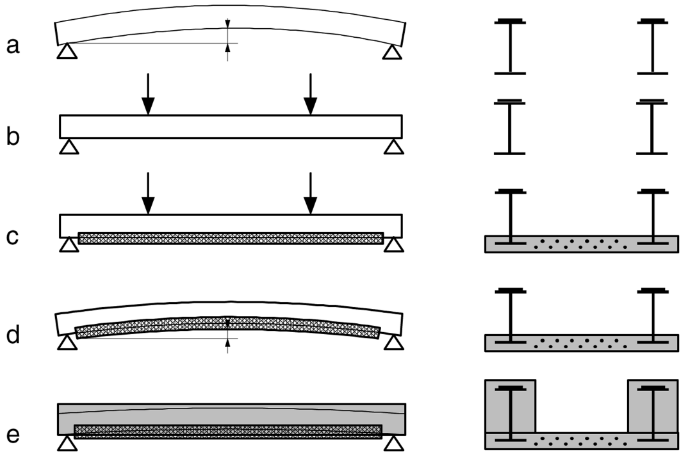

Fig. 2 Construction phases of a U-bridge deck

following day, the remaining naked (upper) parts of the steel girders are enclosed in a 2nd phase concrete (grade C60) to complete the webs of the deck (Fig. 2e). This kind of deck has been designed, among other reasons, to minimize the construction depth, to shorten the erection time on site and to maximize the fatigue resistance.

2. Computation models

Prestressing is transferred at an early age (40 hours or 62 hours) and at high stress levels (around 0.5 $f_{c,\ cube}$) on high strength concrete (concrete should have reached $f_{c,\ cube}$ = 45 MPa at the age of transfer). The composite character of the construction, with the association of the steel of the girders (S355), the steel of the prestressing tendons (grade 1840 MPa) and the two-phases concreting should also be noted. Nearly 400 of these bridge decks have now been constructed since ten years and seem to perform according to expectations.

They have all been designed by TUCRAIL, the engineering office for the high speed railway lines in Belgium. The design model used for service-load limit state verifications is a simple classical computation method where the time-dependent effects of concrete are taken into account within the framework of a pseudo-elastic analysis with variable modular ratios. The modular ratios (m=steel modulus of elasticity/concrete modulus of elasticity) are computed according to an empirical formula given in the Belgian Standard NBN5 (NBN5, 1987):

- m=5.59 after transfer of the prestressing force from the tendons whatever the curing process (instantaneous value);
- m=9.05 for permanent loads (long-term value);
- m=4.97 for variable loads (instantaneous value).

It should be noted that with this method, shrinkage effects are not explicitely taken into account. Moreover, the tension loss by relaxation in the tendons at long-term is not computed but supposed to be 15%.

In the first part of this research, we had the opportunity to monitor during three years the time-dependent evolution of the concrete and steel strains of a 26 m instrumented bridge deck of this kind belonging to a multi-span viaduct constructed in June 2000 at the entrance of Brussels South Station (Staquet *et al*. 2001, 2002a). We have shown that:

- the modular ratio method gives a rather poor prediction of the concrete and steel strains of the instrumented bridge deck;
- the evolution of the creep and shrinkage of the concrete casted in the instrumented bridge deck, assessed through a large series of laboratory tests, is well reproduced by the prediction model from the CEB-FIP Code 1990 in its version published in 1993 and 1999 (fib-CEB-FIP, 1999);
- a far better - although not perfect - agreement with the measured strains is obtained by applying the step-by-step method of time-dependent analysis; this method, which is detailed e.g. in the book by Ghali and Favre (2002), takes explicitely into account the creep and the shrinkage of the concrete, and the relaxation of the prestressing steel.

It may therefore be supposed that the variability of camber that has been qualitatively observed may be partly linked to the application of the simple design model that uses only modular ratios. We had then the opportunity to analyse statistically the camber at the transfer of prestressing and at long-term of a series of 36 bridge decks belonging to the same viaduct and all produced by the same precast yard.

The purpose of the present paper is to report in detail on this statistical analysis carried out in order to extract the pertinent parameters that can improve the accuracy of the predictions provided by each method. The deflections of the bridge decks have been computed by the classical modular ratio method (NBN5, 1987), and also by the step-by-step method. They are compared here with deflections observed in situ at prestressing and at long-term. Note that camber means an upwards permanent deflection.

3. Description of the sample

3.1. Bridge decks geometry, construction and loading

Besides the geometry of the cross section given by Fig. 1, a full description of the 36 bridge decks may be found in Appendix. Seven different types of decks, differing by their span or by their steel girders, have been identified (Table A.1.). Then, each bridge deck differs from the others by its individual early history of construction and concrete strength (Table A.2.).

3.2. Concrete strength

The mix design of the concrete (1st and 2nd phases) is nominally identical for all the decks. The composition is as follows:

- Sand (from Maas river, 0/5): 715 kg/m³
- Aggregates (crushed limestone, 7/14): 1140 kg/m³
- Portland cement (CEM I 52.5 R LA, ASTM III and class 3 CEB-MC90): 380 kg/m³
- Total water: 137 liters/m³
- Water reducing admixture (Visco 4): 7 kg/m³.

Table A.2. gives for each bridge deck the history of its curing, the age of concrete at prestressing (*C* or *ta*1), the concrete strength at prestressing (*A*) and at 28 days (*B*). Concrete strength was measured on cubic specimens with 150 mm sides.

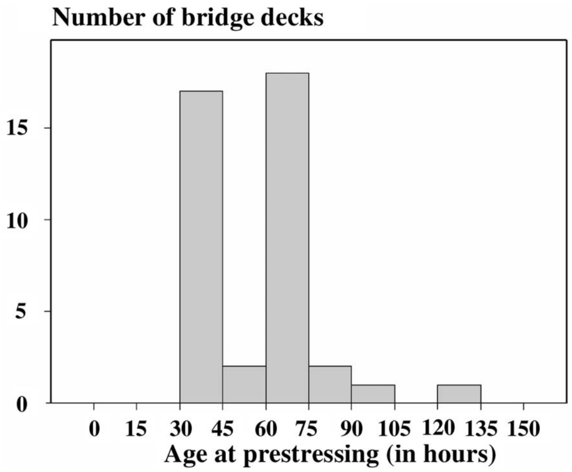

Fig. 3 Histogram of the distribution of the ages of the 1st phase concrete at prestressing

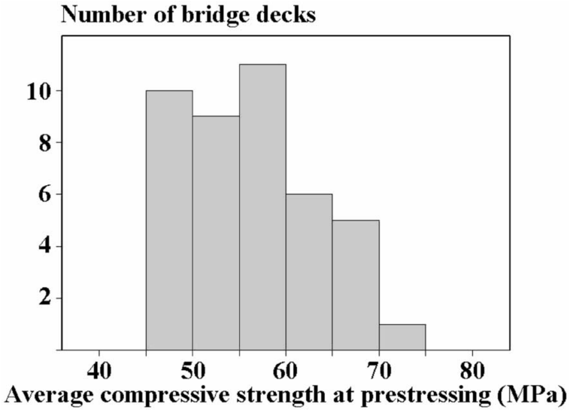

Fig. 4 Average compressive strength of the 1st phase concrete at prestressing (MPa)

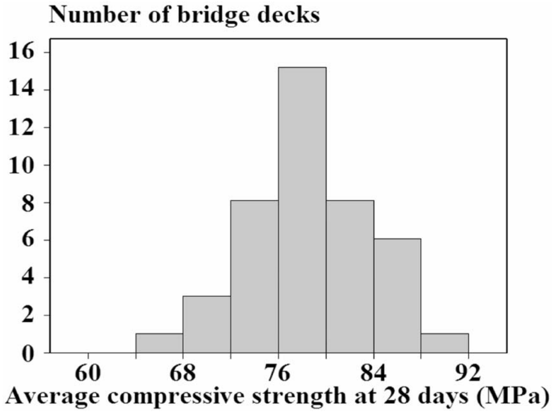

Fig. 5 Average compressive strength of the 1st phase concrete at 28 days (MPa)

The histogram given in Fig. 3 shows the distribution of the ages (in hours) of the concrete from the slab (1st phase concrete) when the preflexion of the steel girders is released and the prestressing force from the tendons transferred. As mentioned previously, two peaks are visible: the first peak occurs at 40 hours for the bridge decks heated at 45°C during the first day after mixing and the second one at 62 hours corresponds mainly to non-heated bridge decks. The minimal, mean and maximal values for the age of concrete at prestressing are: 30, 60 and 126 hours.

The histogram given in Fig. 4 shows the variation of the average cube compressive strength of the 1st phase concrete at prestressing. Fig. 5 shows the variation of the average cube compressive strength of the concrete at 28 days. The minimal, mean and maximal values of the compressive strength at prestressing and at 28 days are [46, 55.8, 74.5 MPa] and [67.5, 78.3, 88 MPa] respectively. The standard deviation of the compressive strength at prestressing is 7 MPa and at 28 days, 5 MPa.

In order to understand exactly the influence of heat curing on the compressive strength of the 1st phase concrete at prestressing and at 28 days, an analysis of variance was made which consists in a test on the equality of the mean values for both groups: without heat curing and with heat curing. The compressive strength at 28 days is found slightly linked to the application of heat curing with a *P*-value equal to 0.055. The *P*-value is the probability that there is no difference between the mean values. Usually, a difference between the mean values can be considered as significant when P-value < 0.05. However, for the compressive strength at prestressing, no difference was found between heated or non-heated first phase concrete (*P*-value equal to 0.828). For the next statistical analysis, the data were divided in two groups: the first group for non-heated concretes and the second group for heated concretes. The box plot given by Fig. 6 shows the average compressive strength of the 1st phase concrete at prestressing for both groups. The mean values and the standard deviation for the non-heated and heated concretes at prestressing are [56.2, 6.4 MPa] and [55.7, 7.4 MPa] respectively. However, the box plot given by Fig. 7 shows that the distribution of the average compressive strength of the 1st phase concrete at 28 days depends on the curing method: curing at 20°C or heat curing at 45°C. The minimal, mean and maximal values of the average compressive strength of the first phase concrete at 28 days without and with heat curing are respectively [75, 80.5, 88 MPa] and [67.5, 77, 87 MPa]. The standard deviations without and with heat curing are 3.7 and 5.3 MPa. The scatter of the results at 28 days is thus larger for concrete with heat curing than for concrete without heat curing.

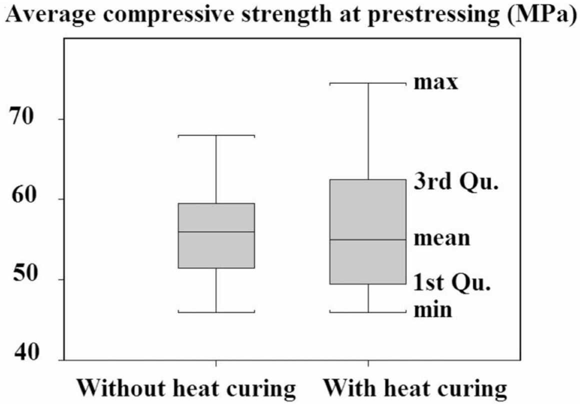

Fig. 6 Compressive strength of the concrete at prestressing (MPa)

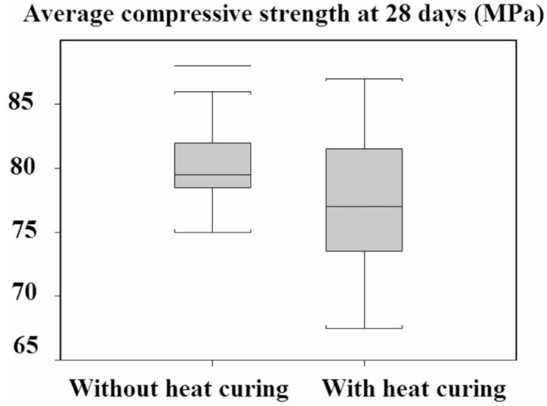

Fig. 7 Compressive strength of the concrete at 28 days (MPa)

3.3. Other variables

Another variable parameter linked to the concrete and suspected to have an influence on the camber at prestressing is the ratio between the stress in the concrete slab at the bottom fiber at mid-span and the compressive strength of the 1st phase concrete at prestressing. This variable is also given in Table A.2 (column *F*). A statistical analysis was made for all the data. The minimal, mean and maximal values of the ratio (in %) found for the sample are [23.5, 39.5, 50.2%] and the standard deviation is 6.4%. For a part of these bridge decks, this ratio reach rather high values, especially if we remember that the concrete compressive strength is measured on cubes. In fact, for this ratio, all the data can be divided in two sets according to the type of the steel girder: hot-rolled (groups 1 to 5 from Table A.1.) or welded (groups 6 and 7 from Table A.1.). The box plot given by Fig. 8 shows that the mean value of this ratio is higher for bridge decks with welded steel girders (44.7%) than the mean value for bridge decks with hot-rolled steel girders (37.5%).

Three others continuous variables suspected to be significant have been selected and reported in

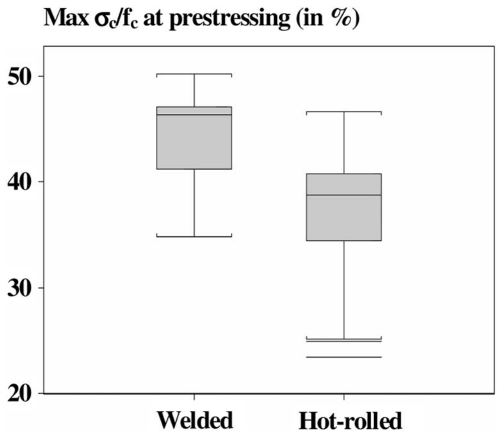

Fig. 8 Ratio between the stress in the concrete at the bottom fiber at mid-span and the compressive strength at prestressing (in %)

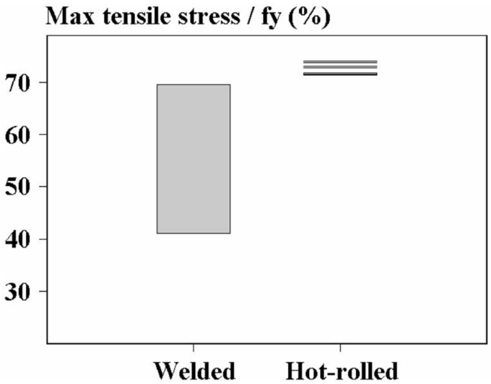

Fig. 9 Ratio between the maximal tensile stress in the steel girders at preflexion and the yield strength (in %)

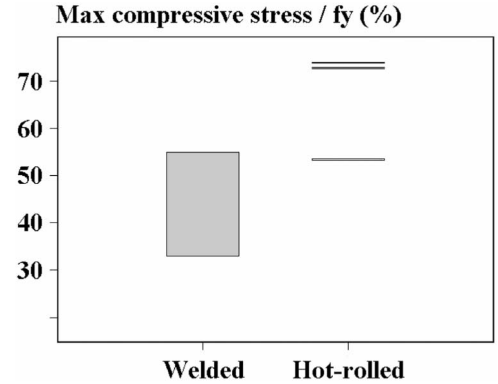

Fig. 10 Ratio between the maximal compressive stress in the steel girders at preflexion and the yield strength (in %)

Table A.2. The first one is the ratio between the maximum tensile stress in the steel girders at the preflexion and the yield strength (column *D* in Table A.2). The minimal, mean and maximal values are [41.1, 67.1, 74%]. But all the data can be divided again in two sets according to the type of the steel girder (Fig. 9). The minimal, mean and maximal values of this ratio for the group of welded steel girders and for the group of hot-rolled steel girders are respectively [41.1, 50.6, 69.5%] and [71.2, 72, 74%]. The standard deviations for the group of welded steel girders and for the group of hot-rolled steel girders are 14 and 0.8 %. This ratio depends thus strongly on the type of steel girders.

The second continuous variable is the ratio between the maximum compressive stress in the steel girders at the preflexion and the yield strength. The minimal, mean and maximal values in percentage were [32.9, 53.4, 74%]. If the data are once again divided in two groups according to the type of the steel girders (Fig. 10), the minimal, mean and maximal values are respectively [32.9, 40.2, 54.9%] and [53.2, 57.2, 74%]. This ratio is also strongly dependent on the type of the steel girders.

The third continuous variable, given by column *E* in Table A.2, is the ratio between the external bending moment due to prestressing and the sum of the external bending moment due to preflexion and prestressing. The minimal, mean and maximal values of this ratio in percentage are [55.9, 71.7, 79%] and the standard deviation is equal to 6.2%.

4. Analysis of the camber just after the transfer of prestressing

4.1. Scope

For this sample of 36 bridge decks with simply supported spans ranging from 18.9 m to 26 m, the cambers measured at mid-span just after the transfer of the prestressing (*da*) and the cambers predicted by the classical design method according to NBN5 (*d*1) and by the step-by-step method (*d*2) have been computed and are given in Table A.2. Computed deflections are obtained by numerical integration of the curvature evaluated in selected cross sections situated along the span. Since the time-dependent effects are not significant at this stage, a very good agreement between measurements and computations should theoretically be obtained. This is not the case, neither with the NBN5 method nor with the step-by-step method. A statistical analysis of the variability of the relative difference between measured and computed camber at prestressing cleared the problem (Staquet *et al.* 2002b).

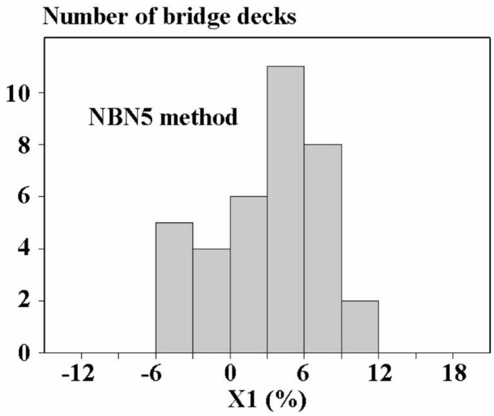

Fig. 11 Distribution of the statistical variable $X1$

4.2. Application of the modular ratio method

Let $X1$ be the relative difference between the camber measured at prestressing (da in Table A.2) and the corresponding camber computed by the NBN5 method ($d1$ in Table A.2).

$$X1 = \frac{da - d1}{da}$$

The histogram shown in Fig. 11 shows the distribution of the statistical variable $X1$. The minimum, mean, maximum and standard deviation values of the distribution are respectively [-5.69, 3.20, 11.19, 4.80%].

In order to explain the variability of the relative difference ($X1$) between measured and computed camber by the NBN5 method just after prestressing, a statistical analysis of the sample was made by using the following continuous or discrete variables:

- A: the bottom slab concrete strength at the age of prestressing transfer;
- presence or absence of heat curing (discrete variable H);
- B: the bottom slab concrete strength at 28 days;
- C: the age of concrete at the transfer of prestressing;
- the type of steel girder (discrete variable T: hot-rolled or welded);
- presence or absence of strengthening plates on the upper flange of the steel girders (discrete variable R);
- D: ratio maximum tensile stress in the steel girders at the preflexion / yield strength;
- E: ratio bending moment due to prestressing / (bending moment due to preflexion+bending moment due to prestressing);
- F: ratio maximum compressive stress in the first phase concrete / concrete strength at the transfer of prestressing.

An analysis of the principal components was made for the continuous variables A, B, C, D, E and F in order to detect in the correlation matrix the pertinent variables to consider in the linear regression models. Table 1 shows the results of the correlation matrix. The most significant variables are D, B and C.

Fig. 12 that shows the correlation between the continuous variables confirms it. Variable $X1$ is strongly correlated to variable D. The continuous variables B, C, D and one discrete variable, namely the type of

Table 1 Results of the correlation matrix between the variables *A*, *B*, *C*, *D*, *E*, *F* and *X*1

	A	*B*	*C*	*D*	*E*	*F*
*X*1	0.07	**–0.255**	**0.218**	**0.47**	–0.09	–0.166

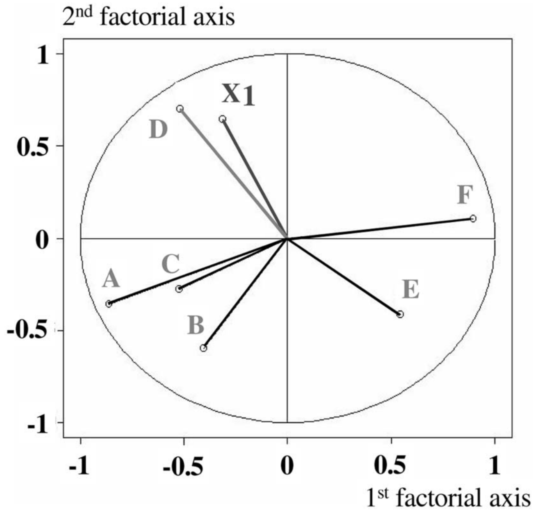

Fig. 12 Representation of the continuous variables *A*, *B*, *C*, *D*, *E*, *F* and *X*1

Table 2 Results of the linear regression model with the variables *B*, *C*, *D* and *X*1

*X*1	*B*	*C*	*D*
P-value	0.058	0.077	**0.002**

Table 3 Results of the linear regression model with the variables *B*, *C*, the type of girder and *X*1

*X*1	*B*	*C*	Type of steel girder
P-value	0.033	0.068	**0.005**

the steel girders (*T*), were considered in a linear regression model in order to explain the variability of the camber at prestressing. The *P*-value is the probability that the variable is not significant in order to explain the variability of *X*1. Usually, the *P*-value can be considered as significant when $P\text{-value} < 0.05$. Tables 2 and 3 show the results for two simulations.

The ratio between maximum tensile stress in the steel girders at the preflexion phase and the yield strength (variable *D*) and the type of steel girders (variable *T*) are the most significant variables to explain the variability of *X*1. If the steel girder is hot-rolled and if the ratio tensile stress/yield strength is high, then the difference between the measured and computed cambers just after prestressing is high too. For a maximum tensile stress higher than 70% of the yield strength, the yield strength can be exceeded locally due to the presence of residual stresses. Furthermore, the hot-rolled steel girders are bent just after rolling. They contain more internal stresses than the welded steel girders. So, the influence of the construction process of steel girders on the camber is significant due to its influence during the elastification phase.

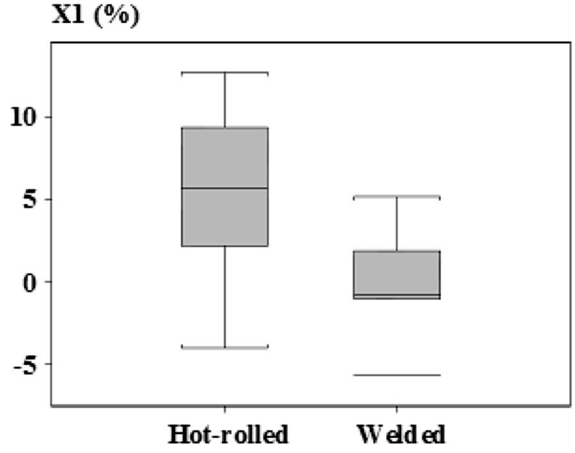

Fig. 13 Box plot of ($X1$) in function of the type of steel girders

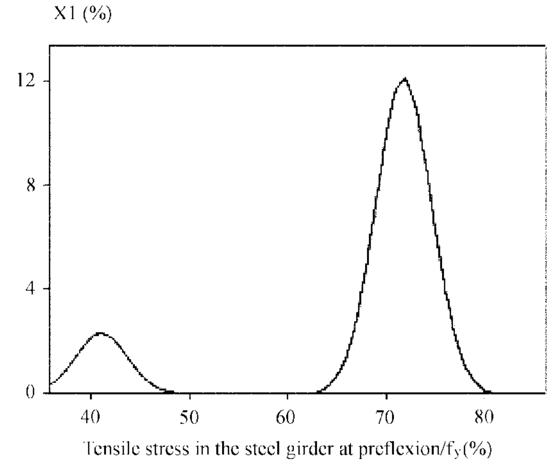

Fig. 14 Box plot of ($X1$) in function of the [tensile stress in the steel girder/yield strength] ratio in %

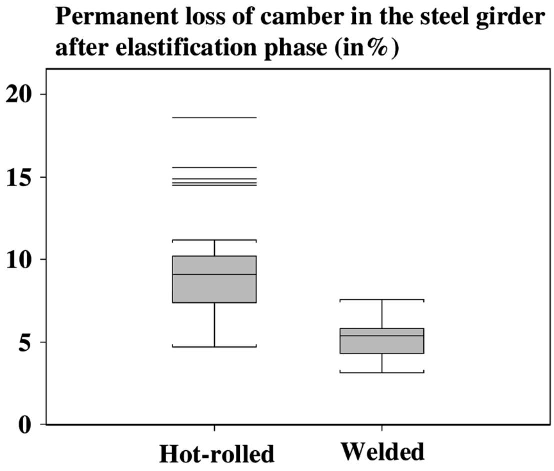

Fig. 15 Measured loss of camber in the hot-rolled and welded steel girders after elastification

The box plot given in Fig. 13 shows the variability of the camber at prestressing in function of the type of the steel girders. The mean values of the variable $X1$ for the bridge decks with hot-rolled steel girders and with welded steel girders are respectively 5.57% and –0.35%.

Fig. 14 shows the variability of the camber in function of the ratio maximum tensile stress/yield strength. The results of the previous statistical analysis are confirmed. In consequence of these results, the loss of camber by elastification must be taken into account according to these two variables in the design process of such composite structures.

The box plot given in Fig. 15 confirms these conclusions. The measured permanent loss of camber in the steel girders after the elastification phase (column G in Table A.2) is higher for hot-rolled girders (mean value: 9.68%) than for welded steel girders (mean value: 5.21%).

We concluded that these two variables should be explicitly introduced in the computation of the camber when using the NBN5 method to design such composite structures. It is not surprising that the age of concrete at prestressing and the heat curing have no influence on the variability of $X1$. Actually, the

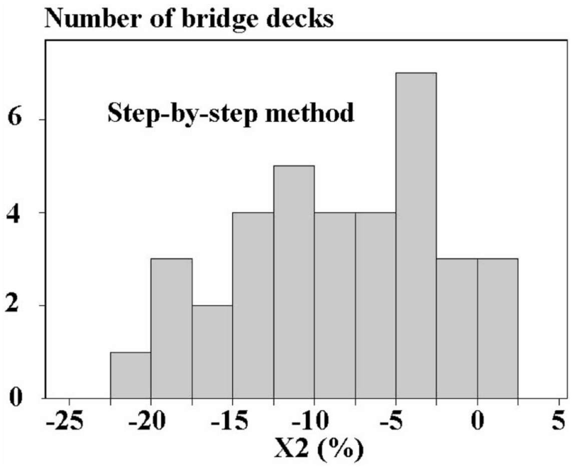

Fig. 16 Distribution of the statistical variable *X*2

NBN5 method uses the same modulus of elasticity or more precisely the same value m=5.59 for the modular ratio- for the 1st phase concrete at prestressing in all situations, whether bridge decks are heated or not, prestressed at 40 or at 62 hours. The specifications required only a minimal concrete strength ($f_{c,cube}$ = 45 MPa) at the transfer of prestressing. In our case, we may surmise that the maturity at 40 hours of the heated concretes was equivalent to the maturity at 62 hours of the non-heated concretes since no difference was found for the compressive strength at prestressing between heated and non-heated concretes.

4.3. Application of the step-by-step method

Let now *X*2 be the difference between the camber measured at prestressing (*da* in Table A.2) and the corresponding camber computed by the step-by-step method (*d*2 in Table A.2).

$$X2 = \frac{da - d2}{da}$$

The histogram shown in Fig. 16 shows the distribution of the statistical variable *X*2. The minimum, mean, maximum and standard deviation values of the distribution are now respectively [-20.98, -8.59, 1.92, 6.18%].

In order to explain this variability, a statistical analysis was made using the following continuous or discrete variables:

- *B*: the concrete strength at 28 days;
- *C*: the age of concrete at the transfer of the prestressing;
- *G*: measured loss of camber in the hot-rolled and welded steel girders after elastification;
- *H*: heat curing;
- *T*: the type of steel girders (hot-rolled or welded).

An analysis of the principal components was made for the continuous variables (*B*), (*C*) and (*G*) in order to detect in the correlation matrix the pertinent variables to be entered in the linear regression model. Table 4 shows the results.

The most significant variable is *C*. Fig. 17 depicting the correlation between the continuous variables

Table 4 Results of the correlation matrix

	B	*C*	*G*
*X*2	0.0099	0.589	-0.13

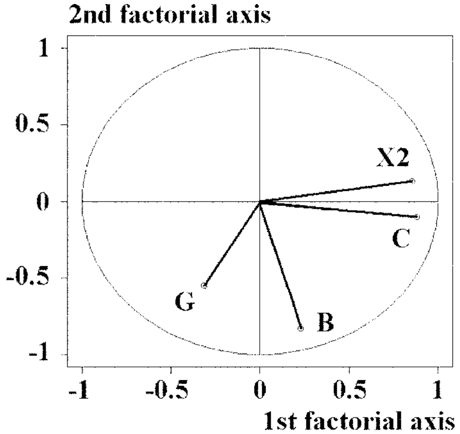

Fig. 17 Representation of the continuous variables *B*, *C*, *G* and *X*2

Table 5 Results of the linear regression with *C*, *H*, *T* and *X*2

*X*2	*C*	*H*	*T*
P-value	0.0268	0.0290	0.0146

confirms it. The variable *X*2 is strongly correlated with the variable *C*.

The continuous variable *C* and the discrete variables *H* and *T* were considered in a linear regression model in order to explain the variability of *X*2. The *P*-value is the probability that the variable is not significant in order to explain the variability of *X*2 and is considered as significant when less than 0.05. Table 5 gives the results.

When using the step-by-step method for computing the camber, the type of steel girders *T*, the age of concrete at the transfer of prestressing *C* and the heat curing *H* can be all considered as significant variables to explain the variability of the camber at prestressing. It is not surprising that the type of steel girders is significant, since it has already been detected in the previous statistical analysis for the NBN5 method. The box plot given in Fig. 18 shows the variability of *X*2 in function of the heat curing. The mean values of the variable *X*2 for the bridge decks without heat curing and with heat curing are respectively −3.98% and −10.75%.

The box plot given in Fig. 19 shows the variability of *X*2 in function of the age of concrete at prestressing. The mean values of the variable *X*2 for the bridge decks prestressed after 2 days or before 2 days are respectively −3.98% and −11.17%.

The step-by-step method takes explicitly into account the value of the modulus of elasticity of concrete at prestressing. This modulus is evaluated by means of the CEB-MC90 (1993) and consequently, it depends on the age of the concrete and the curing temperature by means of an equivalent time. The heated bridge decks are submitted to a temperature of 45°C during the first day after the casting of the slab. To improve the accuracy of the computed cambers by the step-by-step

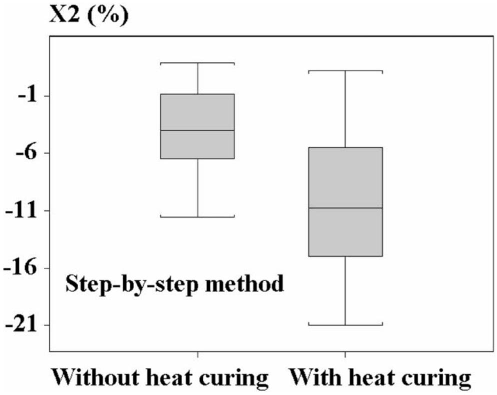

Fig. 18 Box plot of $X2$ with or without heat curing

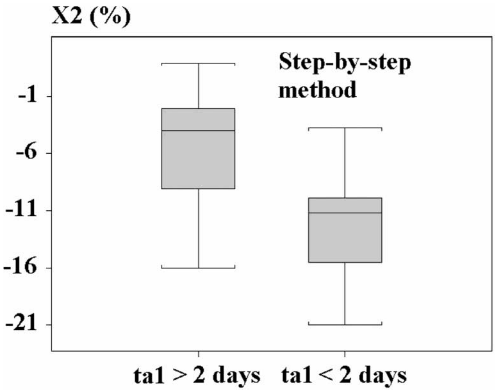

Fig. 19 Box plot of $X2$ for prestressing after or before 2days

method at prestressing, it is necessary to reconsider more finely the evolution of the temperature in the concrete of such composite structures during the first days. The equivalent time used in the computations does not seem to provide a correct estimation of the actual maturity of the concrete in case of heat curing of the bridge decks.

5. Analysis of the camber at long-term

5.1. Scope

The long-term cambers at mid-span of the 36 bridge decks have been computed under permanent loading conditions. They are now compared with long-term cambers measured between 1 and 2 years according to the available data (Staquet *et al.* 2003). In addition to the self-weight, the bridge decks are submitted to the following permanent loads uniformly distributed:

- equipment: 2950 N/m applied at 130 days;
- first part of the ballast: 33060 N/m applied at 272 days;
- second part of the ballast: 12020 N/m applied at 306 days.

5.2. Application of the modular ratio method

$X3$ is the relative difference between the long-term measured camber (*db* in Table A.2) and the long-term camber computed by the NBN5 design method (*d*3 in Table A.2).

$$X3 = \frac{da - d3}{db}$$

The histogram given in Fig. 20 shows the statistical distribution of the variable $X3$. The minimum, mean, maximum and standard deviation values of the distribution are respectively [-67.68, -41.55, -16.66, 11.62%].

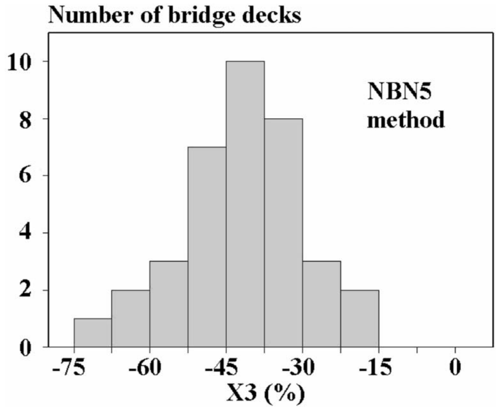

Fig. 20 Distribution of the statistical variable $X3$

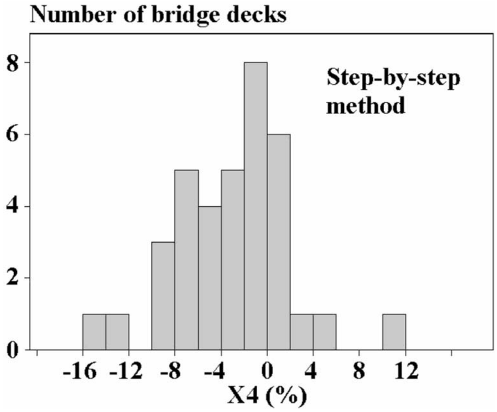

Fig. 21 Distribution of the statistical variable $X4$

Clearly, this computation method does not provide a good estimation of the long-term camber. The NBN5 method simply uses a variable modular ratio to take into account the time-dependent effects. At long-term, creep and shrinkage of concrete have a significant effect on the camber. It is necessary to evaluate more finely the time-dependent effects of concrete and specially the stress redistribution between concrete and steel to improve the accuracy of the predictions at long-term. For that purpose, the step-by-step method was used in the framework of this research.

5.3. Application of the step-by-step method

$X4$ is now the relative difference between the long-term measured camber (*db* in Table A.2) and the long-term camber computed by the step-by-step method ($d4$ in Table A.2).

$$X4 = \frac{db - d4}{db}$$

The CEB-model code in its version 1993 is used for the computation of the creep and the shrinkage effects of the concrete ($f_{c,28}$=64 MPa). The external relative humidity is set to 70% for each computation step. The histogram given in Fig. 21 shows the statistical distribution of the variable $X4$. The minimum, mean, maximum and standard deviation values of the distribution are now respectively [-15.86, -2.99, -10.81, 5.09%].

A statistical analysis has confirmed that the type of girders, the age at prestressing and the type of curing, all variables that had a strong influence at prestressing ($X2$) have no significant influence on the variability of $X4$. In fact, at long-term, creep and shrinkage of concrete are the parameters that affect the most significantly the value of the camber. It is necessary to choose the prediction model for creep and shrinkage that best represents the behaviour of the actual concrete. In these simulations, the selected model has been validated by comparison with test results made in the laboratory. Moreover, the actual history of loading (taking into account temporary support conditions before placement of the bridge decks on their final supports) has been taken into account very accurately in the step-by-step method. At long-term, the step-by-step method provides a rather good agreement between the predicted values and the measurements.

6. Conclusions

An attempt has been made to explain the observed variability of the differences between measured and computed cambers of a new kind of composite bridge deck. The construction of this kind of bridge deck is rather complex with preflexion of precambered steel girders, prestressing of a concrete slab and two-phases concreting. No less than ten variables, including the concrete compressive strength, the influence of heat curing, the type of steel girder (hot rolled or welded) and the age of concrete at prestressing were selected and handled in a statistical analysis of the variability of the camber computed by two different methods for a sample of 36 bridge decks.

The camber just after prestressing was first considered. It was found that neither the variability of the concrete compressive strength (above the minimum required by the specifications) at prestressing, nor the variability of the concrete compressive strength at 28 days can explain the variability of the camber.

For the classical modular ratio design method, the parameters that have the strongest influence on the variability are the ratio between the maximum tensile stress in the girders at the preflexion and the tensile stress and also the type of steel girders. All this is linked to the fact that the internal stresses in the girders before the beginning of the elastification phase are higher in hot rolled steel girders than in welded steel girders. Actually, the measured permanent loss of camber in the steel girders just after the elastification phase is higher for hot-rolled girders than for the welded steel girders.

For the step-by-step computation method, the type of steel girders and the variables linked to the correct estimation of the modulus of elasticity at prestressing (through the influence of the variables heat curing and age of concrete at the transfer of prestressing) are the parameters that affect significantly the variability. The modulus of elasticity was estimated here by the model from the CEB-FIP Model Code. In order to improve the accuracy of prediction of the camber at prestressing, the computation method should take into account more accurately the evolution of the actual concrete strength in the bridge decks for the heated ones and the also the influence of type of steel girders on the loss of initial precamber of the steel girders just after the elastification phase.

For the same sample of bridge decks, long-term cambers have been computed by both methods and compared with measurements in situ. The values of the camber computed by the step-by-step method have been found statistically in far better agreement with the observations than the values computed by the design method using modular ratios. The pseudo-elastic analysis with modular ratios yielded at long-term systematically lower camber values than observed. The accuracy of prediction of the long-term behaviour by the step-by-step method could be improved by taking into account more accurately the influence of heat curing on the time-dependent properties of concrete like its modulus of elasticity and strength, its creep and its shrinkage.

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Appendix

Table A.1 Common characteristics of the groups of decks

N	L (m)	T	R	As (cm²)	Is (cm^4)	v' (cm)	v (cm)	$P1$ (N)	$P2$ (N)	$C1$ (N/m)	$C2$ (N/m)
1	21.5	HEA 1000	YES	411	697451	40	62	16632000	560500	25347	23040
2	21.9	HEA 1000	YES	411	697451	40	62	17028000	551900	25361	23040
3	21	HEB 1000	NO	400	644700	50	50	14256000	616500	25176	23215
4	18.9	HEA 1000	NO	347	553800	49.5	49.5	10692000	603000	25128	23393
5	20	HEB 1000	NO	400	644700	50	50	11880000	649800	25088	23215
6	26	Welded	YES	650	1110634	48	54	22968000	490600	25043	23645
7	24.5	Welded	YES	614	1300188	44	63	16236000	782300	24865	22513

Note: *As* and *Is* are given for one girder only

Notation

N : group number
L : span (m)
T : type of steel girder (hot-rolled or welded)
R : reinforcement of the upper flanges of the steel girders (yes or no)
As : cross section of the girder (cm²)
Is : second moment area of the girder (cm^4)
v' : distance from the centroïd of the girder to its top fibre (cm)
v : distance from the centroïd of the girder to its bottom fibre (cm)
$P1$: total prestressing force applied on the deck (N)
$P2$: value of each load applied at ¼ and ¾ of the span during the preflexion phase (N)
$C1$: loading corresponding to the weight of the 1st phase concrete at mid-span (N/m)
$C2$: loading corresponding to the weight of the 2nd phase concrete at mid-span (N/m)

Table A.2 Individual characteristics of each bridge deck

N	*A*	*H*	*B*	*C*	*D*	*E*	*F*	*G*	*ta* 1 (*d*)	*da*	*d*1	*d*2	*ta* 2 (*d*)	*tb* (*d*)	*db*	*d*3	*d*4
1	57.5	no	78	63	0.72	0.73	37.4	7.4	2.63	65	57.8	64.8	4	517	67	48	71.9
1	54.5	yes	75.5	40.5	0.71	0.73	39.3	6.5	1.69	60	57.8	66.2	3	513	72	48	73.9
1	59.5	yes	81.2	40	0.71	0.73	35.4	6.1	1.67	55	57.8	65.6	3	373	65	48	67
1	62	no	88	62.5	0.72	0.73	35.2	6.5	2.60	61	57.8	63.7	4	370	62	48	67.3
2	56	yes	85.5	40.5	0.72	0.74	39.3	5.9	1.69	62	59.5	67.3	3	450	68	48.2	68.4
2	65	no	76.5	126	0.72	0.74	34.4	9.8	5.27	62	59.5	63.2	6	442	61	48.2	66.8
2	48.5	no	78.5	62.5	0.72	0.74	44.9	9.3	2.60	67	59.5	65.7	4	412	70	48.2	71.3
2	53.5	yes	74.5	40	0.72	0.74	40.5	11.2	1.67	65	59.5	68.6	3	408	72	48.2	71.1
2	53.5	no	80	62.5	0.72	0.74	40.8	6.0	2.60	65	59.5	65.5	4	405	67	48.2	69.9
2	57	no	78.8	86.5	0.72	0.74	38.7	8.0	3.60	62	59.5	64.3	5	391	67	48.2	67.6
2	47.5	yes	76	40	0.72	0.74	45.7	10.0	1.67	65	59.5	67.4	3	387	68	48.2	67.3
2	53.5	yes	76	54	0.72	0.74	41.0	6.0	2.25	65	59.5	66.9	2.5	368	67	48.2	69
2	74.5	yes	82	102	0.72	0.74	30.2	6.5	4.25	65	59.5	64.2	4.5	365	66	48.9	65.7
2	46	yes	69.5	69.5	0.72	0.74	47.6	9.1	2.90	65	59.5	67.4	5	510	71	48.9	71.6
2	54.5	yes	73	40	0.72	0.74	39.7	8.0	1.67	60	59.5	69	2	506	72	48.2	71.9
2	62.5	yes	78.5	62	0.72	0.74	35.4	9.1	2.58	65	59.5	66.6	4	503	68	48.9	69.2
2	46	yes	67.5	40	0.72	0.74	46.6	8.5	1.67	63	59.5	69.8	3	499	72	48.2	73.8
2	47	yes	73.5	40	0.72	0.74	46.0	8.7	1.67	62	59.5	68.9	3	492	68	48.2	72.1
2	49.5	yes	75	40	0.72	0.74	43.8	10.2	1.67	62	59.5	68.7	3	485	71	48.2	71.4
2	60.5	yes	76	63	0.72	0.74	36.5	10.0	2.63	65	59.5	68.6	4	478	68	48.2	71.4
2	55	yes	81.5	62.5	0.72	0.74	40.4	4.7	2.60	63	59.5	66.1	4	475	67	48.9	68.4
2	49.5	yes	84.5	40	0.72	0.74	44.4	10.8	1.67	60	59.5	67.5	3	471	66	48.2	70
2	65	yes	81	62.5	0.72	0.74	34.1	7.1	2.60	57	59.5	66.1	4	468	64	48.9	67.6
2	66	yes	85.5	30	0.72	0.74	33.1	9.2	1.25	60	59.5	69.3	2	464	69	48.2	69.6
3	63	yes	87	62.5	0.73	0.68	34.5	15.6	2.60	67	66.2	73.4	4	461	63	54	71.3
4	50	yes	79.5	54	0.74	0.64	36.5	18.6	2.25	50	51.1	57.1	3	445	57	43.1	56.1
5	51	yes	79.5	40	0.73	0.64	38.1	14.5	1.67	57	59	67.6	3	436	62	49.5	66.6
6	68	no	86	86.5	0.41	0.79	34.8	3.2	3.60	69	68.7	71.7	5	433	82	48.9	73.1
6	49	yes	79	40	0.41	0.79	47.1	5.8	1.67	65	68.7	77.6	3	429	80	48.9	75.3
6	56	no	79.5	62.5	0.41	0.79	41.2	3.6	2.60	70	68.7	74.5	5	426	75	48.9	78.4
6	55.5	yes	72.5	40	0.41	0.79	41.1	5.0	1.67	65	68.7	78.6	3	422	76	48.9	77.4
6	46	yes	84	40.5	0.41	0.79	49.1	5.7	1.69	68	68.7	76.7	3	380	76	48.9	74
6	50.5	no	75	62.5	0.41	0.79	45.3	4.8	2.60	71	68.7	75.1	4	377	81	48.9	79.5
7	51.5	no	79	62.5	0.70	0.63	50.1	5.9	2.60	80	80.6	89.2	4	419	86	64.5	94.1
7	55	yes	70	40	0.70	0.63	46.4	5.1	1.67	85	80.6	93.4	3	415	88	64.5	94.3
7	55.5	no	82	62.5	0.70	0.63	46.7	7.6	2.60	80	80.6	87.2	4	384	78	64.5	90.4

Notation

N	: group number
A	: bottom slab concrete strength at the age of prestressing transfer (MPa)
H	: heat curing (yes or no)
B	: bottom slab concrete strength at 28 days (MPa)
C	: age of concrete at the transfer of prestressing (hour)
D	: ratio maximum tensile stress in the steel girders at the preflexion / yield strength
E	: ratio bending moment due to prestressing / (bending moment due to preflexion + bending moment due to prestressing)
F	: ratio maximum compressive stress in the 1st phase concrete/ 1st phase concrete strength at the transfer of prestressing (%)
G	: measured loss of camber after the elastification phase (%)
*ta*1	: age of concrete at the transfer of prestressing (day) to compute the short-term deflection
*ta*2	: age of the 1st phase concrete (slab) at the casting of the 2nd phase concrete (webs) (day)
da	: measured short-term camber (mm)
*d*1	: short-term camber computed by the NBN5 method (mm)
*d*2	: short-term camber computed by the step-by-step method (mm)
tb	: age of concrete (day) to compute the long-term camber
db	: measured long-term camber (mm)
*d*3	: long-term camber computed by the NBN5 method (mm)
*d*4	: long-term camber computed by the step-by-step method (mm)