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LONG –TERM PERFORMANCE OF LARGE-SPAN PRESTRESSED CONCRETE BRIDGES

Summary

Long-term performance of prestressed concrete bridges is affected by material aspects, construction method, statical system and by many factors, such as creep, differential shrinkage, shear effects, arrangement of prestressing in construction stages and many other.
Prestressed concrete bridges are very sensitive to long-term increase of deflections. This phenomenon has paramount importance for serviceability, durability and long-time reliability of such bridges. Therefore a reliable prediction of deformations of bridges during their construction as well as during their service life is extremely important. The principal significance of this factor for bridge design practice consists in that the results of the appropriate solutions and the developed analytical and design methods help creating the sufficient theoretical tools for reliable and economic structural design of prestressed concrete box girder bridges, enabling great economy of materials, energy and costs and their better utilisation and offer objective and effective tools increasing in the same time the level of durability and efficiency of prestressed concrete box girder bridges. The achieved results enable not only to avoid excessive deflections resulting in long-time serviceability problems, but also possibly other serviceability impairments.

Souhrn

Dlouhodobé působení předpjatých mostů je podmíněno souborem faktorů jako jsou materiálové vlastnosti, způsob výstavby, statický systém apod. včetně vlivu dotvarování, diferenčního smršťování, smykových účinků, účinností předpětí v různých stavebních stádiích a řadou dalších. Existuje řada význačných mostů vykazujících nadměrné a s časem vzrůstající průhyby. Výsledky dlouhodobých měření deformací získaných na řadě předpjatých komorových mostů, jejich podrobné vyhodnocení a porovnání s výsledky teoretické predikce za použití pokročilých matematických modelů jsou stále předmětem zkoumání řady prací. U starých mostů je velice obtížné získat úplné a zcela spolehlivé údaje jak o použitých materiálech, vlivech vnějšího prostředí, postupu výstavby, tak o realizaci měření průhybů – např. časové údaje o tom, kdy se začalo měřit, teplotní vlivy a úroveň, k níž jsou průhyby vztaženy.
Vývoj průhybů ovlivňuje řada faktorů – všeobecně i méně známých [3], [4]; jedním z nich je vliv diferenčního dotvarování a diferenčního smršťování betonu [1], [2], vliv předpětí, vliv smykového namáhání, které mohly být v predikci při návrhu podceněny. Je prokázáno [1], že zejména diferenční smršťování může být příčinou oddálení nárůstu průhybů do mnohem pozdějších období – do stáří mostu, v němž se běžně předpokládá, že deformace jsou již dávno ustáleny.
Hlavní zásady pro dlouhodobé působení předpjatých mostů bez problémů jsou: navrhovat konstrukce robustní (odolné proti nejistotám a málo citlivé na změnu parametrů), volit vhodně poměry rozměrů a dalších charakteristik konstrukce, konstrukce vybavit dalšími možnostmi pro dodatečné předpínání pro případ poruch nebo vyššího zatížení, výpočty založit na nejnovějších poznatcích a výsledcích výzkumu, respektovat skutečné podmínky materiálové, geometrické, časové a okolního prostředí, monitorovat existující konstrukce během výstavby a provozu, a získat tak poznatky pro použití v nových projektech.

Klíčová slova:
most z předpjatého betonu, beton, konstrukce, spolehlivost, trvanlivost, dlouhodobé působení, nadměrné průhyby, předpětí, dotvarování, smršťování, diferenční smršťování, smykové účinky, smykové ochabnutí, nové technologie, monitorování mostů

Key words:
prestressed concrete bridge, concrete structure, reliability, durability, log-term behaviour, excessive deflections, prestressing, creep, shrinkage, differential shrinkage, shear effects, shear lag, new technology, bridge monitoring

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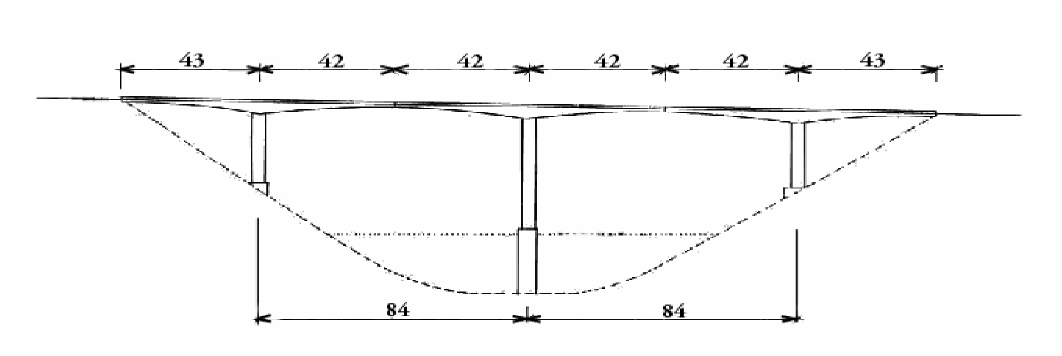
1. Introduction

Prestressed concrete bridges are very sensitive to long-term increase of deflections. In particular, large bridges (exceeding 100 m span) exhibit a gradual increase of deflections during a very long time of service life (even after more than 30 years) in many cases. This phenomenon is very significant for serviceability, durability and long-time reliability of such bridges. Therefore a reliable prediction of deformations of bridges during their construction as well as during their service life is extremely simportant.
Based on the survey of bridges (Tab. 1) it may be stated that the bridges cast in situ by the cantilever method suffer from the progressive growth of deformations more than other kinds of prestressed bridges. This long-term growth of deflections has been observed particularly at the structures with a hinge at the midspan, but the experience shows that the same phenomenon occurs at the continuous bridges, although the absolute deflection values are usually lower.
Almost identical time variations of the deflection line have been observed at many bridges in various countries. Thus, it may be concluded that the growth of deflections even after decades is not a result of local conditions (kind of aggregate, cement type, environmental effects, labour, etc.) and that it is necessary to look for the reasons in the overall arrangement of the bridge, in material characteristics and in the structural performance of the bridges.
There are many reasons for the deflection increases, which usually are coupled together. The prestressed structures are extremely sensitive to the deflections in general. The deflection is a result of two opposite actions: the first one is represented by the external (vertical) loadings as dead load and live load. The other one, which has the opposite direction, is the effect of prestressing. The both mentioned actions when acting separately would produce individually significant deflections of *opposite* directions. The resulting deformation due to simultaneous action of the both loadings - due to external (vertical) loads and due to the prestress - is, however, the *difference of the mentioned deflections*.

Tab. 1. Examples of bridges exhibiting excessive deflections

No.	Country	Bridge	Span [m]	Structural system	Completion
1	Czech republic	Zvíkov - Vltava	84	Hinges at midspans	1963
2	Czech republic	Zvíkov - Otava	84	Hinges at midspans	1962
3	Czech republic	Želivka	102	Hinges at midspans	1968
4	Czech republic	Teplice	53	Segmental	1983
5	Czech republic	Dìèín	104	Continuous beam	1985
6	Czech republic	Mìlník	140	Continuous beam	1993
7	Czech republic	Klabava	55	Segmental	1993
8	UK	Cogan	95	Segmental	1988

9	UK	Grangetown	72	Segmental	1987
10	USA	Parrots-Ferry	195	Continuous, light concrete	1978
11	Switzerland	La Lutrive	131	Hinges at midspans	1973
12	Switzerland	Chillon	104	Hinges at midspans	1970
13	Switzerland	Fegire	107	Continuous beam	1979
14	Switzerland	Paudeze	104	Continuous beam	1972
15	Sweden	Stenungsund	94	Hinges at midspans	
16	Sweden	Tunsta	107	Hinges at midspans	1955
17	Sweden	Alno	134	Hinges at midspans	1964
18	Sweden	Kallosund	107	Hinges at midspans	1958
19	Norway	Nordsund	142	Hinges at midspans	1971
20	Nederland	Wessem	100	Continuous beam	1966
21	Nederland	Maastricht	112	Continuous beam	1968
22	Nederland	Grubbenvorst	121	Continuous beam	1971
23	Nederland	Empel	120	Continuous beam	1971
24	Nederland	Heteren	121	Continuous beam	1972
25	Nederland	Ravenstein	140	Continuous beam	1975



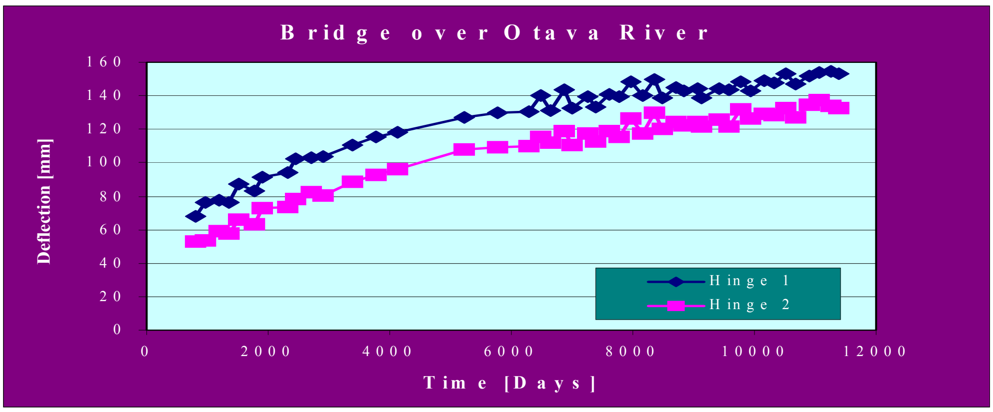


Fig. 2. Bridge across the Otava River a) general arrangement, b) exhibiting excessive deflections

This *difference of large numbers* is very sensitive, a *small change* in one of these numbers may result in very *significant change* of their difference, i.e. change of the final deflection value.
In the reality, all the incoming parameters are of random nature. The dead and live loads are usually known rather reliably. On the other hand, the prestressing shows much larger deviations from the assumed values. The initial uncertainty yields from unknown prestress losses just in the initial stage of the structure. Further increase of prestress losses depends on many factors, and the losses are not easy to predict, in particular if the tendons are grouted and the stress can vary along the length of the tendon. Regarding the above-mentioned effect of sensitivity of deflections on the contributing components, the randomness of prestressing can play a very significant role when predicting the deflections.
The effect of prestressing also varies in dependence on the shape of the tendon. The straight tendons induce compression and bending in dependence on their eccentricity. In particular, the bending effect is contributing to the magnitudes of deflections, the axial component of prestressing is favourable to eliminate tensile stresses in the section, but it normally has no effect on deflections of the bridge beams. The curved tendons produce significant bending effects and can effectively contribute to balancing the effects of permanent loads. The ratio between compressive and bending effects of prestressing is therefore very important with respect to the deflection development.
In practice, the eccentricity of prestressing tendons is usually assumed in design calculations very simply. The designs are commonly based on the assumption that the amount of steel in prestressed structures is low and its effect on the cross-sectional parameters (position of the centroid and values of the moments of inertia) can be neglected. This leads to the apparently safer design, because of the lower assumed stiffness of the sections. Evaluating the effects of the vertical loads, this approach could be acceptable. However, if the prestressing effects are evaluated, it is necessary to take into account that due to the steel cross-sectional area the centroid of the whole section is closer to the prestressing steel than the centroid of the plain concrete section. Thus, the eccentricity of the tendons is actually smaller.
The prestressing level in the long-term view is significantly affected by shrinkage and creep strains of concrete. The shortening of a bridge beams results in a decrease of the prestressing force. The shrinkage strains are quite often larger than expected and recent investigations also showed that the shrinkage may develop during a very long time and its final value can hardly be estimated from short-term measurements.
Cracking is further significant factor influencing the stiffness of sections. There are numerous reasons for crack occurrence. Initially, during casting the hydration heat induces selfequilibrated stresses, which are rapidly reduced due to relaxation in the initial stage of maturing of concrete. Since the tensile strength is low, some cracking appears, particularly if the surface is exposed to

low temperatures. Later, the drying process and temperature changes also induce additional stresses close to the surface, which often exceed the tensile strength of concrete and result in cracking. These cracks are later usually partially closed because the drying process continues into the core of the elements and induces compression in the surface layers. It should be noted that creep and shrinkage produce or relax the stresses, which cause cracking, and cracking in turn changes the stress state which causes creep. The last but not least factor causing cracking can be a local overstressing of concrete in tension in vicinity of singularities like anchors or supports causing a nonuniform distribution of stress within the cross-section. The cracking may progress due to repeated loading (during construction or during the service life) and reduce the stiffness of the sections. These aspects are usually not taken into account in the design calculations.

Such a reduction of stiffness leads to increased deformation due to the loads, but even more significantly influences the efficiency of prestressing. The eccentricity of prestressing in cracked sections becomes smaller and therefore the action of prestressing reduces with time.

The loading of very young concretes is typical for bridges cast in situ by the cantilever method. Various systems are used in order to reduce the stress concentrations under the anchor plates, but the young concrete is in general sensitive to any load. The effects of mechanical loads are combined with eigenstresses in concrete and there is no guarantee that the internal damage does not occur. The additional factor leading to differences between common analytical predictions and the actual performance is the fact that the cantilever is repeatedly loaded by the cast segments and by prestressing, which produces the cyclic load of the young concrete. Although the mechanical stresses are usually low, the principle of superposition can hardly be applied because of the unloading and reloading and the reversibility of strains.

The analyses applied in the design practice must necessarily take into account all changes of the structural system during the construction process as well as the effects of creep and shrinkage of concrete. The approaches to the structural analysis belong to one of three following levels: (i) usual calculation taking only the action of bending moments into account (i.e. disregarding the shear deformations and thin-walled effects like the shear lag), (ii) a frame analysis taking the shear deformations of webs into account but disregarding the shear lag, (iii) an advanced analysis taking the shear deformations of webs as well as the shear lag into account.

In practice, short-term as well as long-term structural analyses of box girder bridges are typically based on frame idealisation of the level (i) neglecting shear deformations and the shear lag. This can result in underestimation of deflections due to action of *vertical loads. The prestressing effects*, being accompanied by no or minor shears, are, on the other hand, predicted by this elementary bending theory rather satisfactorily. This is why *the concept of effective widths*, if they

are assumed the same for evaluation of effects of vertical loads and prestressing, is in the case of prestressed concrete bridges *completely wrong*.
As it has been mentioned above, the final stress distributions and deformation shapes of box girder bridges are given by a *difference of these values* - due to external (vertical) loads and due to the prestress, i.e. by a *difference of large numbers*. This means, when usual methods of analysis are applied, i.e. combining the underestimated predictions of deflections due to action of vertical external loads (the error of which easily may reach 10 per cent or more, depending on the arrangement of a bridge) on the one hand, and relatively accurate deflections due to prestressing on the other hand, much more higher discrepancy from the real final deflections can be obtained, or even this may lead to final deflections of opposite signs. The error of predictions by methods ignoring the shear effects can thus easily be enormous. Unfortunately, this rather trivial cause is usually neglected in structural analyses for box girder bridges design.
It is characteristic that box girder bridges (like all structures) inevitably have imperfections of various types. Particularly significant are geometric imperfections, i.e. deviations from nominal shape parameters, physical imperfections, i.e. fluctuations of material characteristics, random effects of environmental factors. To the category of imperfections belongs also the degradation processes. Since the imperfections are of random nature, adequate and reliable approaches to the structural analysis have inevitably to be based on stochastic principles.

It may be summarised that the excessive and during time increasing deflections of long-span prestressed bridges are caused by a combination of several simultaneously acting factors having character of material relations, stress distributions and structural performance of a bridge. The research on this problem is extremely important, not only to avoid the excessive deflections resulting in long-time serviceability problems, but also possibly in cracking, corrosion or other serviceability impairments. Such bridges have to be either closed or repaired well before the end of their initially projected design life. The cost to the society is tremendous, and in fact greatly exceeds in strictly economic terms the cost of catastrophic failures due to mispredicted safety margin. It also should be noted that a *wrong prediction of the development of deflections* means that also prediction of the *distribution of internal forces*, particularly in bridges changing the structural systems, can be *quite far from the reality*. Thus, it should be recommended that appropriate methods of analysis are needed to predict a realistic time variation of deflections as well as internal forces redistributions in long span bridges.

2. The aim of research

The research focuses on an objective analysis of causes of excessive deflections of prestressed concrete bridges and their long-term increase which has paramount importance for serviceability, durability and long-time reliability of such bridges. The first step is to understand and explain the causes of the mentioned problems. The principal target consists in that the results of the appropriate solutions and the developed analytical and design methods help creating the sufficient theoretical tools for bridge design practice for reliable and economic structural design of prestressed concrete box girder bridges while respecting random character of input parameters, enabling economy of materials, energy and costs and their better utilisation and offer objective and effective tools increasing in the same time the level of durability and efficiency of prestressed concrete box girder bridges. The achieved results enable not only to avoid excessive deflections resulting in long-time serviceability problems, but also possibly cracking, corrosion or other serviceability impairments.

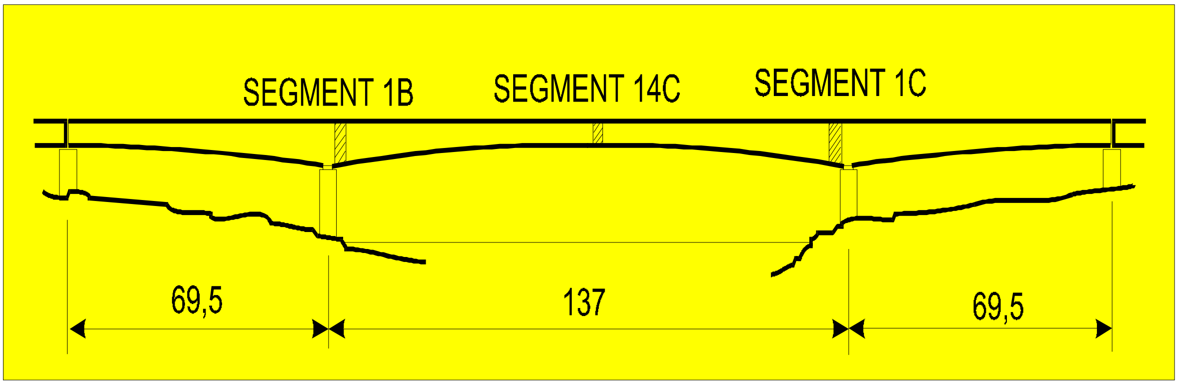
The appropriate solutions are based on reliable experimental data and physical phenomena and take into account randomness of the input parameters. The application of numerical methods and development of new advanced computer software for the analysis of multi-dimensional structures when loaded by both constant or long-term time varying loads is necessary. The developed methods are capable to obtain realistic prediction of the structural performance.

A thorough analysis of experimental data, experience from monitored structures and results of theoretical studies allow to fully understand the problem and to formulate general conclusions about real structural performance of long-span prestressed concrete box girder bridges.

The formulation of general conclusions about prediction of real structural performance of long-span prestressed concrete box girder bridges, about forecasting service conditions and about possibilities of lowering maintenance and repair costs is also the practical objective of the research.

3. Experimental investigation

Prestressed concrete structures often exhibit larger long-term deflections than it was predicted in design calculations. A number of bridges, particularly cast in situ using a cantilever method, were monitored and the time deflection diagrams were recorded. Monitoring of strains and deflections at existing structures provide necessary data for improvement of the design of new structures and also for assessment of older bridges. The data obtained from measurement on structures should be compared with the measured strains on laboratory specimens. The differences, which often indicate smaller long-term strains on specimens than those measured on structures, have to be explained and the appropriate models suitable for structural analysis developed. Although a number of structures have been monitored only a limited amount of valuable information could be obtained if some data are missing. An extensive experimental program has been started on the bridge over the Ohře River in the Czech Republic. The aim of this program was to find and evaluate those factors, which influence the long-term development of deflections and still are not taken into account in practical design of bridges, and to verify the prediction of deflections.



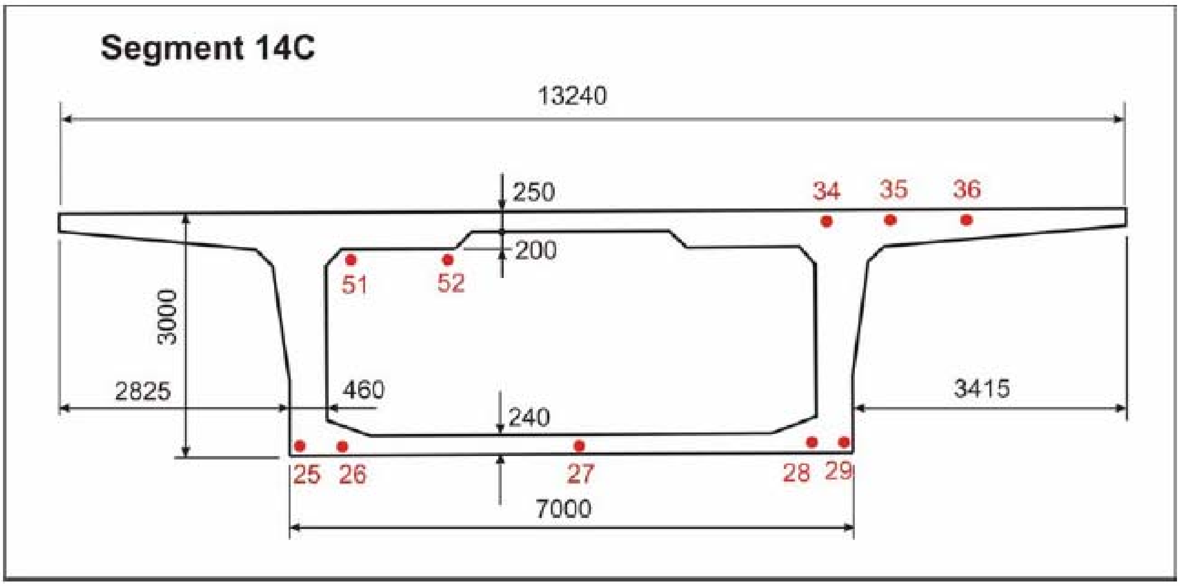


Fig. 3 Bridge across the Ohře river: a) elevation, b) mid-span segment

The bridge across the Ohře river is a typical box girder (Fig. 3). The continuous beam carrying two lanes of the motorway is 13.2 m wide and has 3 spans (69.5 + 137 + 69.5 m). The depth of the box girder varies from 7.2 m over the supports to 3 m at the midspan and at the ends of the bridge, where approaching viaducts are connected. The thickness of webs varies between 0.46 m and 0.6 m. The thickness of the bottom flange at the support –1,3 m is reduced continuously to 0,24 m at the midspan. The bridge was cast by cantilever method in situ; balanced cantilevers were connected at the midspan, then the auxiliary supports in short spans were removed. Since the experimental program was planned earlier than the construction started, the gauges were cast inside the concrete sections.

The strains are measured at the three segments in the main span. Fig. 3 shows the view of the bridge and the position of the instrumented segments. The segments 1B and 1C are located symmetrically, but the age of their concrete differs significantly. The segment 14C is located just at the end of the second cantilever adjacent to the closing joint at the midspan. The vibrating wire strain gauges produced by the company Gauge Technique (U.K.) are either cast in concrete or attached to the mild reinforcement. 12 gauges in one segment were distributed in the top and bottom flanges. In spite of the special care during the construction process four gauges at the segment 14C failed and they were replaced by additional gauges attached to the bottom surface of the top flange.

The measured strains in the top flange of the support segment are plotted in Fig. 4. The initial increase of strain is induced mainly due to the stressing of cables during a construction process. Further increase is observed due to creep and shrinkage of concrete and due to temperature effects. The drop of strain after completion of the bridge (the last three points) has to be verified by the next measurements, but there is an opinion that the drop is induced by temperature changes.

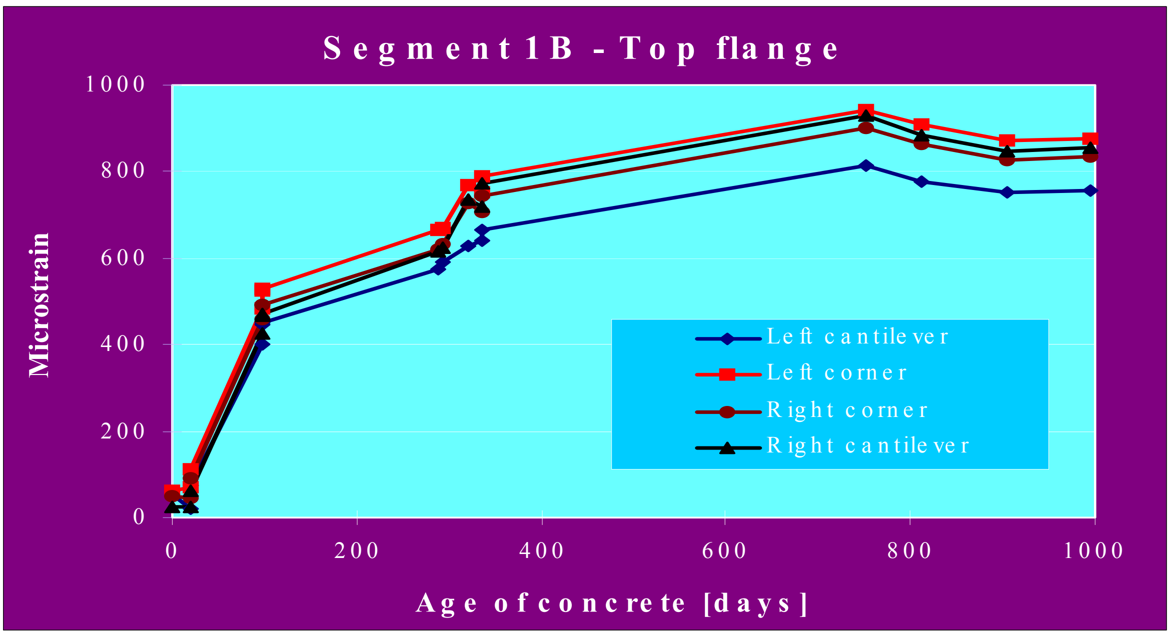


Fig. 4 Measured strains in the top flange of the support segment

The bonded prestressing tendons were assembled typically from 15 ropes of 15,5 mm in diameter. The development of prestressing force was of primary interest. The gauges which measure the stress in prestressing steel using an elastomagnetic method, were located at strands of three typical shapes of segments. Reduction of prestressing was monitored from the time of stressing and it will continue in the future. The last reading taken after the load test showed that the prestress losses in short cantilever cables were about 12-15%, the losses in long cantilever cables varied from 8-12% and the continuity cables at the midspan, which were stressed later, showed the prestress losses only 5-7%. This observation is more favourable than the estimates in the design calculations and it proves the quality of stressing system and the good quality of concrete.
The concrete cylinders were subjected to long-term loading inside the box girder. These specimens provide the information on material characteristics and enable to compare the strains measured on specimens with those measured on the structure. The agreement was not observed at earlier built bridges. Shrinkage of concrete was measured on cylinders without loading. The early results show that the measured strain variation can be approximated by the CEB-FIP model 1990 better than by national code models, but still the measured values are larger than the predicted strains.

4. Structural analysis

In design practice, short-term as well as long-term structural analyses of box girder bridges are usually based on simple frame models (e.g. a continuous beam), taking into account only the action of bending moments (i.e. ignoring shear deformations in webs and thin-walled effects like the shear lag). This fact results in underestimation of deflections due to action of vertical loadings as dead load and live load. The prestressing effects, being accompanied by none or minor shears, are predicted by the elementary bending theory more satisfactorily. The resulting deformation due to simultaneous action of both loadings - due to external (vertical) loads and due to the prestress - is, however, the difference of the mentioned deflections. The difference of large numbers is very sensitive because a relatively small change in one of these numbers can result in very significant change of their difference, i.e. in the final deflection value. This means, when a usual method of analysis, ignoring the shear effects is applied (combining the underestimated predictions of deflections due to action of vertical external loads on one hand, and relatively accurate deflections due to prestressing on the other hand) a very significant discrepancy from the real final deflections can result. It is also obvious that the concept of effective widths is incorrect in the case of prestressed concrete bridges to model the shear lag effects (if in the analysis the same effective width is assumed for evaluation of effects of vertical loads as well as of the prestressing).

4.1 Shear lag

One of the causes of discrepancies between the predicted and observed deflections of prestressed box girder bridges are the effects of the shear lag and shear deformations of webs. This rather trivial cause is usually neglected in structural analyses.

It is well known that shear lag, induced by shear deformations of flanges of box girders, can result in very non-uniform distribution of longitudinal normal stresses across the flange widths, and may also significantly influence the girder deflections. At box girders (particularly with external tendons), where the cross sectional area of webs represents only a small fraction of the total cross sectional area, the shear deformations of webs also significantly participate on deflection magnitudes. Both these phenomena - the shear lag and shear deformations of webs - appear if the box girder is stressed by shear forces. This is the case when a box girder bridge is loaded by external vertical loads (e.g. by self weight and/or by live load). Prestressing by straight (or nearly straight) tendons induces no shear forces (or shears with low magnitudes) and, therefore, there are no (or relatively very low) shear lag and shear deformations of webs.

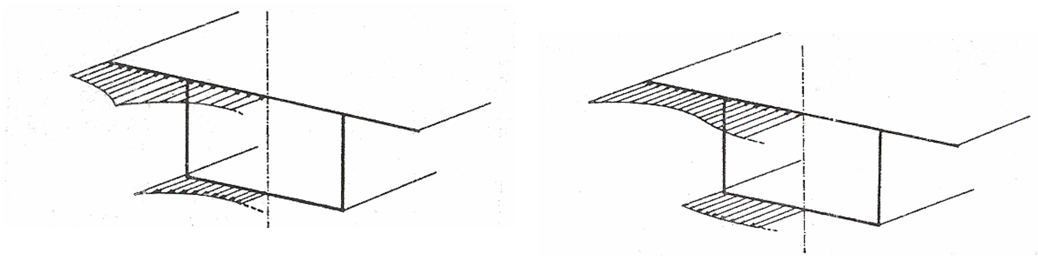


Fig. 5 a) classical shear lag, b) negative shear lag

From the point of view of the overall performance of the bridge the effects of the shear lag and shear deformations of webs have to be assessed. It is well known that shear lag, induced by shear deformations of flanges of box girders, can result in very non-uniform distribution of longitudinal normal stresses across the flange widths, and may also significantly influence the girder deflections (Fig. 5). As the flange width increases in relation to span, the shear lag effects become more pronounced. At box girders, where the cross sectional area of webs represents only a fraction of the total cross sectional area, the shear deformations of webs also significantly participate on deflection magnitudes. Both these phenomena - the shear lag and shear deformations of webs - appear if the box girder is stressed by shear forces. This is typically a case when the box girder bridge is loaded by external vertical loads (e.g. by self weight and/or by live load). Prestressing by straight (or nearly straight) tendons induces no shear

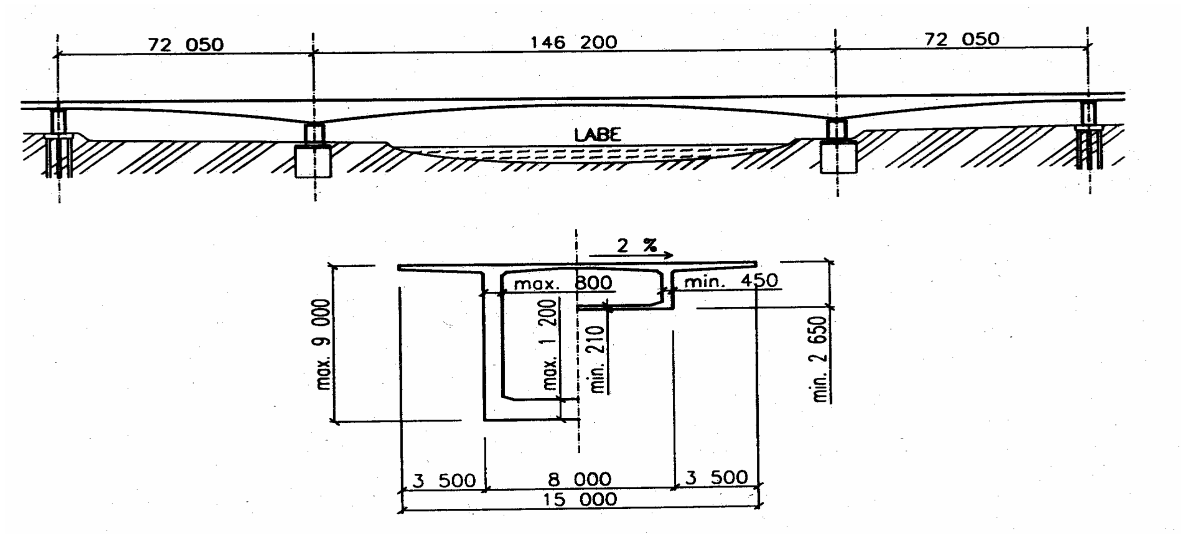
forces (or minor shears) and, therefore, there are no (or relatively very low) shear lag and shear deformations of webs.
In practice, short-term as well as long-term structural analyses of box girder bridges are typically based on simple frame idealisations (like a continuous beam), taking usually only the action of bending moments into account (i.e. disregarding shear deformations and thin-walled effects like the shear lag). This can result in underestimation of deflections due to action of vertical loads. The prestressing effects, being accompanied by no or minor shears, are, on the other hand, predicted by the elementary bending theory more satisfactorily. Thus, it is apparent that the concept of effective widths, if in the analysis the effective width is assumed the same for evaluation of effects of vertical loads as well as of the prestressing, is in the case of prestressed concrete bridges completely wrong.
The deflection is basically a result of the two opposite actions. The first one is represented by the external (vertical) loadings like dead load and live load. The other action, which has an opposite direction, is the effect of prestressing. The both mentioned actions when would act individually will produce significant deflections of opposite directions. The resulting deformation due to simultaneous action of the both loadings - due to external (vertical) loads and due to the prestress - is, however, the difference of the mentioned deflections.
This difference of large numbers is very sensitive and therefore a relatively small change in one of these numbers can result in very significant change of their difference, i.e. in the final deflection value. This means, when a usual method of analysis, ignoring the shear effects is applied, i.e. combining the underestimated predictions of deflections due to action of vertical external loads, (the error of which easily may reach 10 per cent or more, depending on

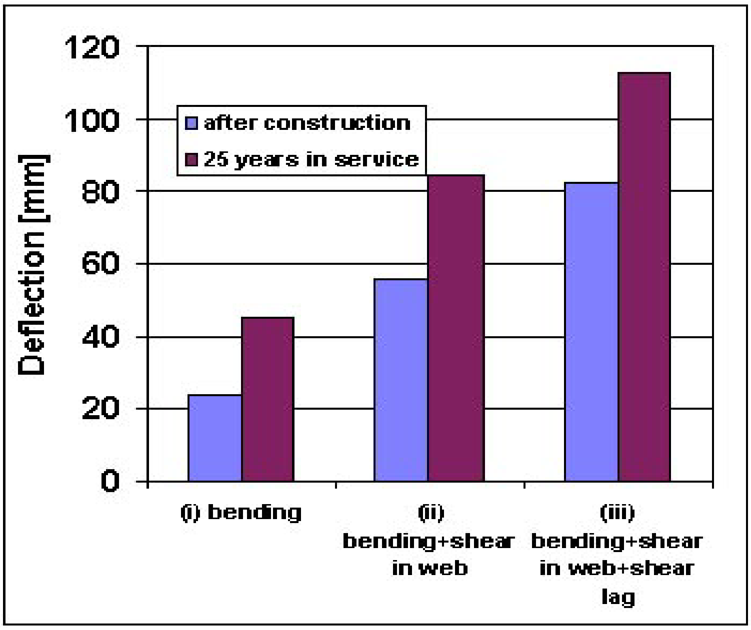
Fig. 6 View of bridge over Labe river at Mělník

the arrangement of a bridge) on the one hand, and relatively accurate deflections due to prestressing on the other hand, a very significant discrepancy from the real final deflections can be obtained.

4.2 Shear effects

As an example, some results of a deterministic analysis of deflection development of the bridge shown in Fig.6 and 7a due to the dead load and prestressing are presented in Fig.7b,c. The results obtained by the analysis performed on three levels are presented comparing: (i) usual calculation taking only the action of bending moments into account (i.e. disregarding the shear deformations and thin-walled effects like the shear lag), (ii) a frame analysis taking the shear deformations of webs into account but disregarding the shear lag, (iii) an advanced analysis taking the shear deformations of webs as well as the shear lag into account. Fig.7b shows the deflections of the end of the cantilever for two times during construction stage, Fig.7c similarly presents the deflections at the main span midspan during the bridge service life. The impact of shear effect on the deflection magnitude and their time development is seen to be very significant.





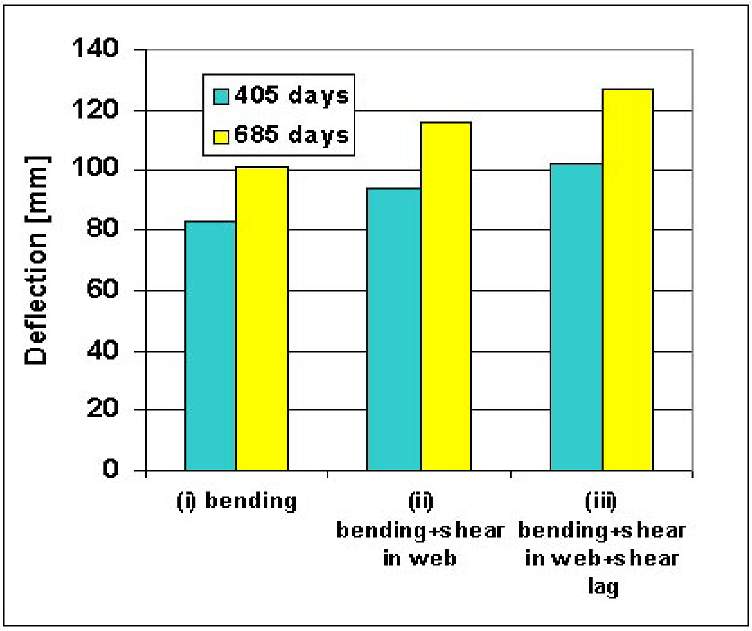


Fig. 7 a) Bridge across the Labe river at Mělník, b) increase of cantilever deflections due to shear effects, c) increase of deflections in the final structural system due to shear effects

In the reality, all the incoming parameters are of random nature. The dead and live loads are usually known rather reliably. On the other hand, the prestressing show much larger deviations from the assumed values. The initial uncertainty yields from unknown prestress losses just in the initial state of the structure. Further increase of prestress losses depends on many factors, and the losses are not easy to predict, in particular if the tendons are grouted and the stress can vary along the length of the tendon. Regarding the above mentioned sensitivity of deflections on the contributing components, the randomness of prestressing plays very significant role when predicting the deflections.

4.3 Prestressing

An important question concerning the prestressing arrangement is to study in what manner the long-term deflections of a bridge in *the final structural system* are influenced by the prestressing applied *during the construction stages*. It can be shown that low deflections of the bridge *during the cantilever construction stages* do not automatically results also in low long-term deflections during the bridge service life.

This can be demonstrated on a very simple example – two cantilevers are prestressed by tendons anchored at the end cross-sections as in Fig. 8. This arrangement of prestressing is very efficient to reduce deflections in the cantilever stage, but it is absolutely inefficient to reduce deflection increases after the cantilevers are made continuous to form a final structural system. Such a final system (a continuous beam) deforms in time due to creep as being without any prestressing.

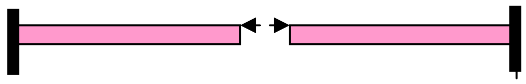


Fig. 8 Model arrangement of the bridge

Results of a parametric study elucidating this phenomenon are presented next. The intention is to investigate efficiency of prestressing in the top flange of a box girder bridge applied in the construction cantilever stage on the deflection increases at midspan of the main field of the girder in the final structural system: a three span continuous box beam as in Fig. 9; the span length of the first and third spans equals 0.6 of the span length of the main – second field. The box girders analysed were tapered, characterized by the ratio *N* of moments of inertia of the support cross-section and the midspan cross-section.

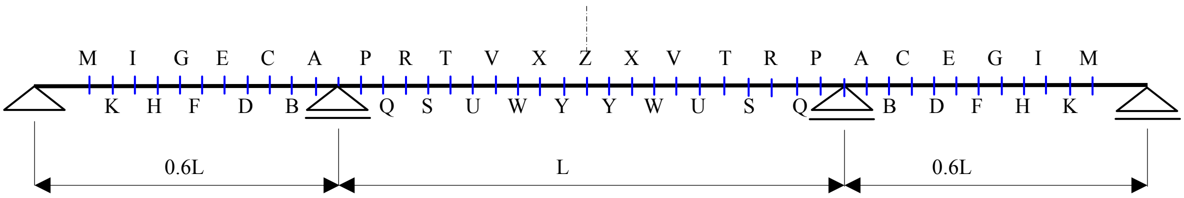


Fig. 9 Model arrangement of tendons positions

The study estimates the influence of a tendon anchored in a cross-section *i* in the first span (*i* = ***A,B,C,....,M***, see Fig. 9) and in a cross-section *j* in the second span (*j* =***P,Q,R,.....,Z***) on the long term deflections of the cross-section *Z* in the final structural system. Taking the effect of a short tendon between cross-sections *A* and *P* as referential (its effect is taken as a unity, see Tab.2), the efficiency of a wide variety of tendon arrangements is summarized (for a tapered box beam, *N* = 20) in Tab. 2.

Tab. 2 Efficiency of tendon arrangements in the dependence on anchor locations

Stiffness ratio *N* = 20		**Tendon anchored in the first span in the cross-section**										
		A	***B***	***C***	***D***	***E***	***F***	***G***	***H***	***I***	***K***	***M***
Tendon anchored in the main span in the cross-section	***P***	**1**	1.638	2.443	3.479	4.832	6.629	8.928	11.625	14.410	16.675	18.397
	Q	1.556	2.194	2.998	4.035	5.387	7.184	9.483	12.180	14.965	17.231	18.952
	R	2.167	2.805	3.610	4.646	5.998	7.795	10.094	12.791	15.576	17.842	19.563
	S	2.825	3.463	4.268	5.305	6.657	8.454	10.753	13.450	16.235	18.501	20.222
	T	3.468	4.106	4.911	5.948	7.300	9.097	11.396	14.093	16.878	19.144	20.865
	U	3.960	4.598	5.403	6.440	7.792	9.589	11.888	14.585	17.370	19.636	21.357

	V	4.000	4.638	5.443	6.479	7.832	9.629	11.928	14.625	17.410	19.675	21.397
	W	3.135	3.773	4.578	5.614	6.967	8.763	11.063	13.760	16.544	18.810	20.532
	X	0.881	1.519	2.324	3.360	4.713	6.510	8.809	11.506	14.290	16.556	18.278
	Y	-2.969	-2.331	-1.526	-0.490	0.863	2.660	4.959	7.656	10.440	12.706	14.428
	Z	-8.341	-7.703	-6.898	-5.862	-4.510	-2.713	-0.413	2.283	5.068	7.334	9.056

It follows from the presented study that the efficiency of the tendon arrangement to reduce long term deflections of a bridge in the final structural system is beneficial if:

- in the construction stage, the tendon in the first span is anchored in a longer distance from the internal support
- in the construction stage, the tendon in the main span is anchored in a distance from the internal support, which depends on the stiffness ratio *N*. The location and the length of this beneficial region vary – for a beam of constant cross-section this region is located between sections *R* and *T*, for a very pronouncedly tapered beam between sections *T* and *W*. The most beneficial anchor locations are shown in Tab.3

Tab. 3 Most beneficial anchor locations

Stiffness ratio *N*	1	5	10	15	20
Location of the anchor	***S***	***T***	***U***	***U***	***V***

The tendons running between cross-sections *D* and *Y* or the tendons between cross-sections *G* a Z are quite inefficient.
It can be also seen that:

- a short tendon situated above support is much less efficient than a longer tendon
- applying a suitable arrangement of anchoring, it is possible to achieve much higher efficiency in comparison with a short tendon situated above support. This efficiency increases with the stiffness ratio *N* – see Tab. 3 where this multiplicator exceeds the value of 20
- an unsuitable arrangement of the tendon locations can result in an opposite effect – such tendons will cause long-time *increase* (instead of reduction) of the midspan cross-section deflections (the all multiplicators with negative signs appearing in the left bottom part of Tab. 1; it typically concerns the tendons anchored in the main span far away from the support and close to the support in the first span)

5. Material aspects

5.1 Prediction of concrete creep and shrinkage

Realistic prediction of concrete creep and shrinkage is a necessary condition for achieving appropriate prediction of deflection variations of concrete bridges; such an analysis has obviously be able to take into account all changes of the structural system during the construction process. Concrete creep and shrinkage are very complex phenomena involving several interacting physical mechanisms on various scales of the microstructure, which are influenced by many variable factors. Therefore, a relatively high degree of sophistication in a realistic prediction model is inevitable. Moreover, a creep and shrinkage prediction model should be capable to updating of its several principal parameters on the basis of short-time tests carried out on the given concrete to be used. The updating task, which is essential for the modern design of large span bridges can be successful only if the creep and shrinkage curves of the model have suitable shapes not only for long time periods but also for short time periods. By updating the model, significant improvements of long-term prediction can be achieved.

An Internet page has been developed, which makes a realistic creep and shrinkage prediction model B3 immediately accessible to any engineer. This Internet page is a design tool, which on filling the boxes for all the data of the concrete, gives values of creep and shrinkage strain as well as the creep coefficient instantly. The use of web page is much more easier than hand calculations practiced by some. The design office has no need to develop a computer subroutine for evaluating the equations of the prediction model. The task does not even require a qualified engineer.

5.2 Effects of differential shrinkage and drying creep

Shrinkage - besides the axial shortening of the bridge beams - can also affect the box girder deflections due to nonuniform development of shrinkage resulting from different thicknesses of flanges.

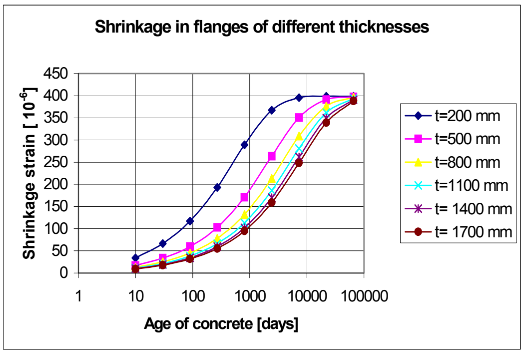
It is well known that the time evolution of shrinkage and the drying-induced part of creep in box girder bridges is considerably affected by the thicknesses of flanges and webs as well as by the environmental conditions. In the negative moment regions over the internal (intermediate) supports of continuous box girders, the shrinkage and creep strains evolve rather differently, on one hand in the thin top flanges and on the other hand in the thick bottom flange plates. Generally, the thin parts of the cross section tend to shrink and creep faster than the thicker parts.

It is also typical that the top and bottom flanges are exposed to different environmental conditions - in the top flange and in parts of the webs a more intensive drying occurs (due to sunshine, etc.), especially during the first stages of the construction process than in the bottom flange. Later, after the completion of the structure, the drying of the top flange is considerably reduced if an asphalt pavement is placed on top of concrete, since the pavement effectively insulates the top surface of concrete.

The different time evolutions of shrinkage and drying creep in the individual parts of the cross section are phenomena that may significantly affect the long-term performance, particularly the long-term bridge deflections. The reason is that the differences in strains in the individual parts of the cross section induce an additional curvature of the deflection curve of the bridge girder.

To document the different development of shrinkage and creep in concrete slab elements, Fig. 8a shows the developments of shrinkage strains for various slab thicknesses predicted by a realistic creep and shrinkage prediction model – Model B3. The differences between the individual shrinkage developments are plotted in Fig.8b, where the thickness of 200 mm is taken as the reference; it is clearly seen that these differences are very significant.

a) b)



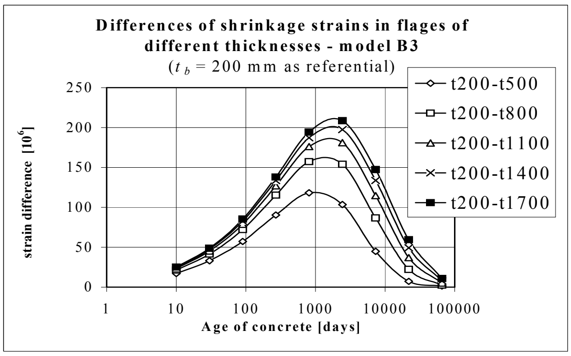


Fig. 8 Differential shrinkage

The difference in the evolutions of creep in the individual parts of the cross section results generally in a coupling between the effects of bending moments and axial forces. An axial force acting at the centroid of the cross section will produce not only axial displacements but also rotations of the cross section increasing with time. Vice versa, an applied bending moment will produce not only rotations but also axial displacements. The significance of these effects depends mainly upon the structural system and geometry arrangement (mainly upon differences in thickness) and upon creep characteristics, but it should be mentioned that actually exists. This phenomenon, however, may effect the lever arms of the prestressing tendons, thus influencing the time evolution of the effects of prestress.The differential creep response, however, varies in the range of several percent only, which is much less significant than the differences in shrinkage.

To be able to asses the effects of the differences in shrinkage and creep and to be able to take them into account when predicting the structural performance of prestressed concrete box girder bridges (particularly their deflections), the simplified shrinkage and creep prediction models which characterize only the overall behavior of the cross section and especially ignore the drying processed

must be abandoned and an appropriate analytical tool considering the different in the rheological properties of the individual parts of the cross section must be applied.
It has been found that the curvatures of a box girder bridge in the cantilever stage first increase over a long period, but then they reach a maximum and afterwards they decrease. The magnitude of deflections strongly depends upon the flange thickness differences. A minor curvature appears also in box girder segments having both flanges of the same thickness, but different widths (because of difference in volume - surface ratio). The maximum point is reached at relatively very old age of concrete. After the maximum point the shrinkage rate of the thick bottom flange becomes greater (eventually much greater) than the shrinkage of the thinner top flange (which has at that time essentially finished its shrinkage). The result is a delay in the onset of significant downward of deflections of box girders, which gets shifted to a much later period than would be expected according to common level of understanding.
In contrast to the differential shrinkage, the differential creep in the flanges of different thicknesses does not play a significant role in the curvature evolution. There are two reasons: the differences in the compliance functions for slabs of different thicknesses are not very significant; and the flexural stiffness of the webs in their own plane restrains quite effectively the flexure of the whole box girder caused by tendency of flanges to creep differently. Differences in the drying creep thus have only a minor effect on box girder deflections.
When investigating performance of a box girder bridge in the final structural system, it is obvious that the deflections of the cantilevers due to the differential shrinkage evolve freely only until their free ends are joined to create the final structural system of the bridge. Box girders of two final structural systems are used in the practice:
The free ends of cantilevers are joined by a hinge. There is no restraint against continuing rotations in the hinge, the vertical force that develops in the hinge is of marginal importance (unless the structure is highly non-homogeneous). Therefore, the deflections of the bridge due to differential shrinkage may continue to evolve almost freely, like the original cantilevers, and no significant secondary internal forces are induced in the structure after the cantilevers are joined.
Box girder cantilevers are joined continuously at their ends. Due to the differential shrinkage accompanied by the differential creep, the originally free end cross sections of the both cantilevers would continue to rotate at the ends after joining. This is prevented after the structure is made continuous - to prevent relative rotation of the end cross sections, bending moment develops in time in the joint.
Preventing changes of the rotations at cantilever box girder of *constant cross section* would have no effect on deflections if the box girder were homogeneous. This means that the differential shrinkage in a *clamped* box girder of *constant cross section* does not induce any changes of deflections after the cantilevers are

joined. This finding is approximately valid also in the case of an internal span of a long multispan continuous girder provided that the girder is erected by the cantilever method simultaneously in all spans.

It may be concluded that the deflection evolution in the final structural system due to the differential shrinkage is significantly reduced after the cantilevers are joined and that the deflections of the internal spans of continuous box girders are much less affected by the differential shrinkage than these of bridges with midspan hinges. This may partially explain why bridges with midspan hinges exhibit a longer period of deflection increases than the continuous girders.

Negligible though the midspan differential shrinkage deflection increments may be in continuous box girders, significant bending stresses may occur in the final structural system after the girder is made continuous. Restraining a free continuing evolution of deformations of the original cantilevers requires large redundant internal forces and moments. This redundant bending moment exhibits a rather complex time evolution, changing from negative to positive values, and may reach significant magnitudes. The moment distribution is almost uniform throughout all the central span length - from the bulky deep cross sections at supports to the very shallow and thin-walled cross sections in the midspan region.

This may cause significant additional stress values, particularly in the midspan cross section. The magnitudes of these stresses must be added to the all the other stress components obtained by the routine calculations according to common design practice.

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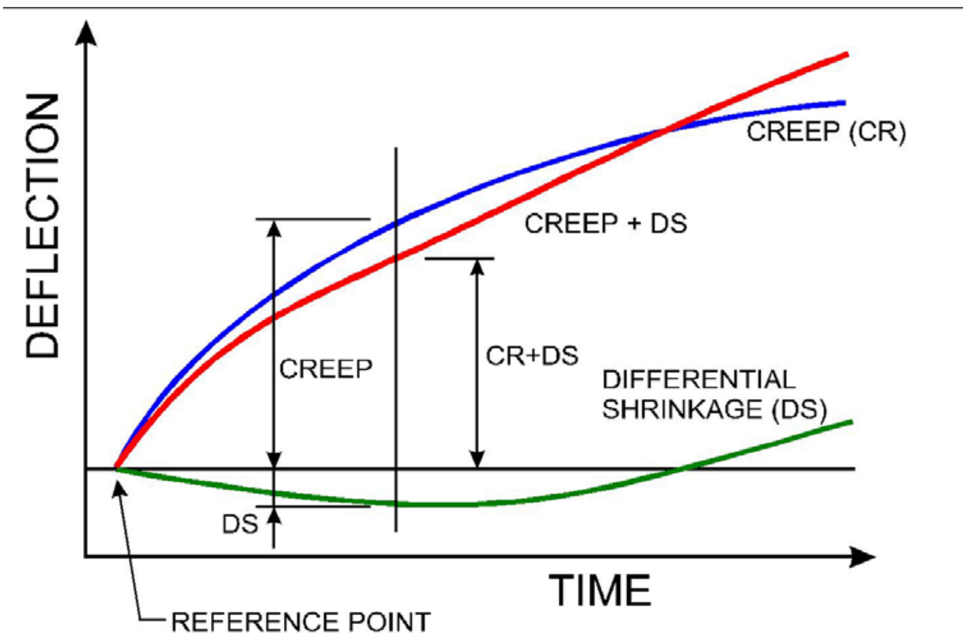
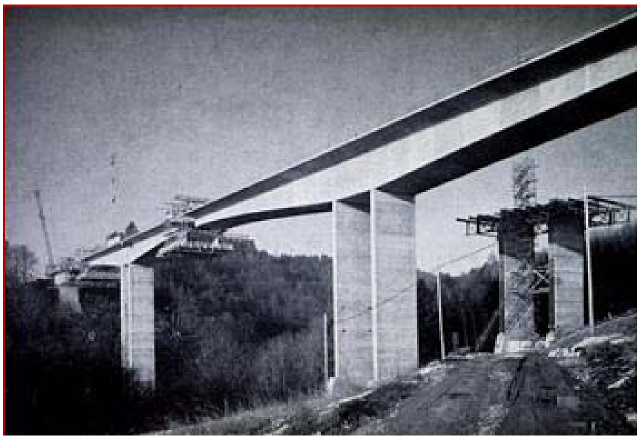
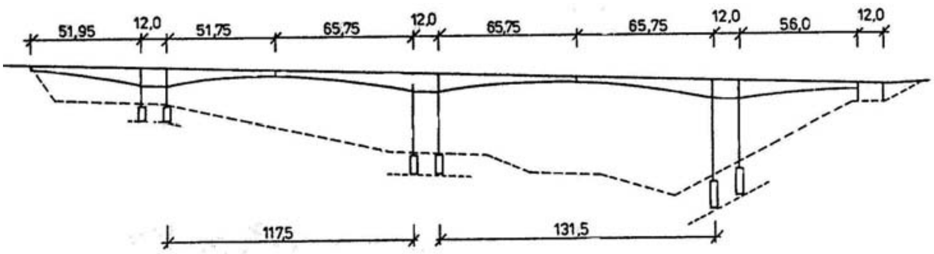


Fig. 9 Delay of deflection development due to differential shrinkage

To understand why these additional stresses may be high, it must be realized that these stresses result from bending moments whose magnitudes depend on overall stiffness of the span to which a large contribution comes from the regions near the supports where the girder is very deep and that the high bending moment is resisted in the midspan region by *a weak cross section.* It is obvious that occurrence of the significant additional stresses in the midspan regions due to the differential shrinkage will intensify with increased size differences between support and midspan cross sections. So it is evident that the stresses in the midspan regions of long span prestressed concrete box girder bridges, calculated without taking the differential shrinkage into account, may be merely fictitiousIt can be concluded that the differential effects, particularly the differential shrinkage, are the cause of the delay of deflections leaving them to develop to much longer ages. Schematically, these interactions are shown in Fig.9. The deflections are reduced in the first period due to the drying shrinkage and latter, they are increased, as it is seen in Fig. 9 in red line.

A bridge taken for a comparative analysis is the *La Lutrive* bridge, built in 1973 in Switzerland, with hinges at midspans. The arrangement of the bridge (the elevation and the cross section shape) is shown in Fig. 10a,b. The midspan deflections gradually increased, as depicted in Fig.11; and in 15 years they exceeded 150 mm.
The maximum deflection at the midspan hinge of the bridge due to the differential shrinkage of about 25 mm is reached at the age of concrete of 1300 days. Fig. 11 shows the diagram of the measured deflection increase of the midspan hinge of the bridge during the period of 11000 days after the start of monitoring of the bridge and developments of deflection increases estimated by:
(i) the analysis applying the B3 model considering the mean cross sectional approach,
(ii) the more realistic analysis applying
the B3 model and simultaneously respecting via B3 model the differential shrinkage and
(iii) the very simple ACI 209R-92 model.
The results correspond to the findings shown in the Fig. 9:
The agreement between the measured deflections and the values obtained applying the B3 creep prediction model is quite satisfactory;
A time lag is seen in the posterior prediction of the deflection evolution respecting the differential shrinkage (the fourth curve in fig. 11 which exhibits the best fit to the measured values) in comparison with the mean approach;
The curves corresponding to the simple ACI 209R-92 model, exhibit very steep initial deflection development and, on the other hand, significantly underestimate the deflection increases in the later periods, thus proving this model to be far from reality.





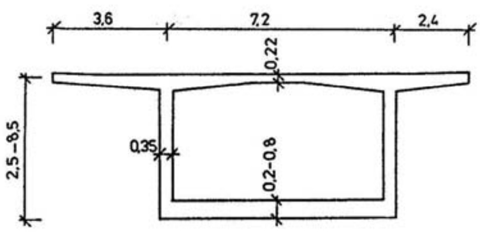


Fig. 10 The *La Lutrive* bridge in Switzerland

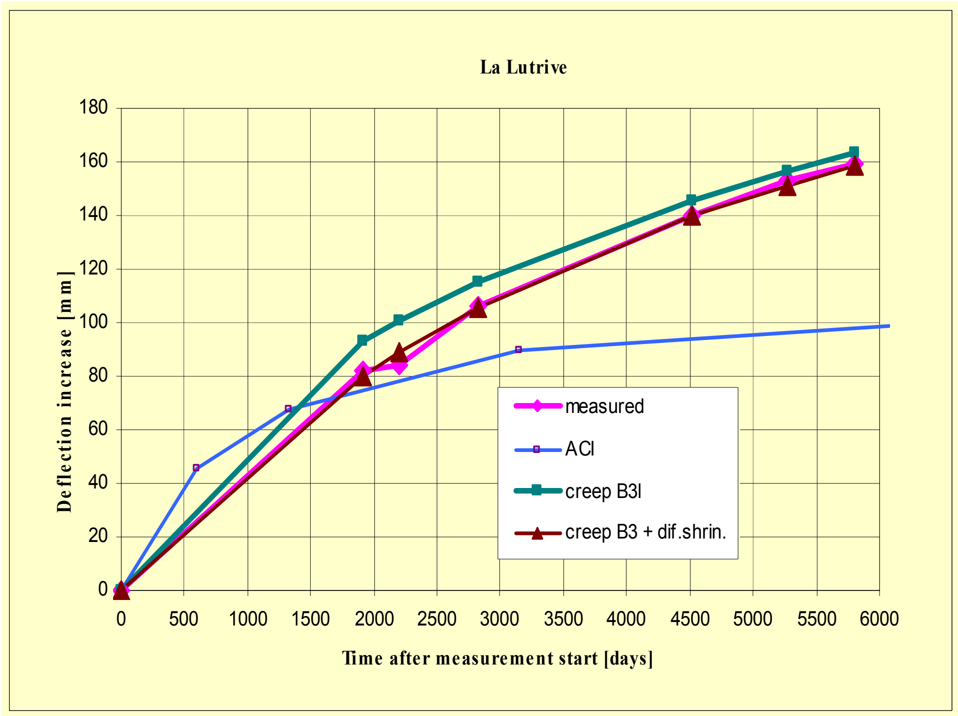


Fig. 11 The *La Lutrive* bridge – deflection development

6. Conclusions

In the reality, all the incoming parameters for predicting the deflection variations of prestressed concrete box girder bridges are of random nature. The dead and live loads are usually known rather reliably. On the other hand, the prestressing exhibits much larger deviations from the assumed values. The initial uncertainty yields from unknown prestress losses just in the initial state of the structure. Further increase of prestress losses depends on many factors, and the losses are not easy to predict, in particular if the tendons are grouted and the stress can vary along the length of the tendon. Regarding the above-mentioned sensitivity of deflections on the contributing components, the randomness of prestressing plays very significant role when predicting the deflections.

It may be summarised that the excessive and time-increasing deflections of long-span prestressed bridges are caused by a combination of several simultaneously acting factors. The research on this problem is extremely important, not only to avoid excessive deflections resulting in long-time serviceability problems, but also possibly in cracking, corrosion or other serviceability impairments. Such bridges have to be either closed or repaired well before the end of their initially projected design life. The cost to the society is tremendous, and in fact greatly exceeds in strictly economic terms the cost of catastrophic failures due to mispredicted safety margin. It also should be noted that a *wrong prediction of the development of deflections* means that also prediction of the *distribution of internal forces*, particularly in bridges changing the structural systems, can be *quite far from the reality*. Thus, it should be concluded that appropriate methods of analysis are needed to predict a realistic time variations of deflections as well as internal forces redistributions in long span prestressed concrete bridges.

A thorough analysis of experimental data, experience from monitored structures and results of theoretical studies will allow to fully understand the problem and to formulate general conclusions about real structural performance of long-span prestressed concrete box girder bridges.

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