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Construction Sequence Effects on Externally Prestressed Composite Girders

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Summary

The authors examine composite steel-concrete beams prestressed by external cable, in order to analyze the influence of the most common construction and prestressing sequences, by evaluating both the initial linear behavior under service load and the nonlinear behavior under increasing load up to failure. The analysis is based on a model, described in previous works, which considers the cable slipping at saddle points and nonlinear constitutive laws for materials composing the structure. In this paper the analysis is extended for considering different construction sequences in a unitary manner and some numerical results on simply supported and continuous two-span girders are reported.

1. Introduction

External prestressing, often used in brigde construction and strengthening, usually employes cables disposed parallel to the steel beam web and anchored at ends. Cable profiles are defined by saddle points at which they can slip with negligible friction.

Generally, stretched cables permit increasing the global load-carrying capacity, however some other advantages derive from their presence. At service conditions maximum deflection can be controlled and notably reduced. In continuous beams concrete deck cracking at intermediate supports can be prevented under a wide range of external loads. The cables undergo small stress increments under service load thanks to the redistribution of traction due to slipping and this determines a further ductility resource at failure.

Substantially different construction and prestressing sequences are adopted in practice [1, 2], leading to different collapse modalities and different stress and strain distributions in materials (concrete, beam steel, reinforcement steel and high-strength cable steel). Such differences have not been sufficiently investigated in the past and this paper intends to show the different behaviors under increasing external loads up to failure for the following three cases.

<u>Case A.</u> The concrete deck is cast on the steel beam supported by a large number of additional supports (propped beam). After the concrete hardening the additional supports are removed and prestressing is applied.

<u>Case B</u>. The concrete deck is cast on the steel beam supported by the final constraints only (unpropped beam). After concrete hardening prestressing is applied.

<u>Case C.</u> Prestressing is applied to the steel beam only, before the concrete deck cast.

In all the previous cases the concrete deck can be separately prestressed before the connection with the lower steel beam, obtaining a further increment of compressive stress to prevent cracking.

In previous works [3, 4], the authors presented an analytical model and a numerical solving procedure which consider the nonlinear consitutive behavior of materials and the slipping of the cable at saddle points.
In this paper analytical and numerical aspects are briefly recalled and attention is focused on construction sequences, by defining a unitary approach and by showing some results of practical interest.

2. Nonlinear analysis for different construction sequences

The model of beam displacements preserves the cross section planarity and the orthogonality with the deformed axis, so that the beam strain $\varepsilon_b(x,y)$ can be deduced from the axis displacements in the longitudinal direction $u_0(x)$ and the axis deflection $v_0(x)$, in the following form

$$\varepsilon_b = u_0' - y v_0'' \tag{1}$$

The cable strain derives from the deformation of the whole beam. Denoting by the vector Q_i with components (x_i, y_i), the initial positions of the intermediate saddle points (i=1..D-1) and anchorages (i=0, i=D), and denoting by $\Delta_i(\varphi)$ the difference of the values assumed by a generic function φ between the saddle points i and i-1, the cable strain assumes the following expression

$$\varepsilon_c(u_{0i}, v_{0i}, v_{0i}') = \frac{1}{\Lambda}\sum_{i=1}^{i=D} \alpha_i\left[\Delta_i(u_0) - \Delta_i(y v_0')\right] + \beta_i \Delta_i(v_0) \tag{2}$$

where $\Lambda = \sum_{i=1}^{i=D} |Q_i - Q_{i-1}|$ is the cable length in the reference configuration; α_i and β_i are geometrical terms related to the cable profile. The expression is linearized coherently with the assumption of small displacements and rotations. The nonlinear elastic constitutive laws of beam materials and cable can be posed in the form:

$$\sigma = F(\varepsilon - \varepsilon_0) \tag{3}$$

where ε_0 is the residual strain present at the reference configuration.
The equilibrium condition is determined by means of the Virtual Work Principle. The numerical solution of the nonlinear problem has been performed by approximating the diplacement fields with shape functions and solving the system of equilibrium equations by using the iterative Newton-Raphson method, [3, 4].

The nonlinear analysis prevents superimposing stresses at every construction step, nevertheless the analysis remains sufficiently simple by identifying the correct residual strain to consider at each construction stage. In the sequel the configuration at which the stress is null at all the points of the steel beam, which is present at each construction stage, is assumed as the reference configuration. So that, in all the anlyses the following condition is considered

$\varepsilon_0 = 0$ in steel beam.

The residual strains in concrete deck, reinforcements and cable must be determined in each stage of the different construction sequences.

Case A - propped beam prestressed after the concrete deck cast

The case of propped beam is the simplest situation because no preventive analysis is required for evaluating the nonlinear behavior under increasing loads. In the case of cast in situ concrete deck, the residual strain is null in the whole beam while the residual strain in the cable ε_{0p} permits controlling the prestressing traction force. The effects due to a preventive prestressing applied to the upper concrete deck before the connection with the steel beam can be analyzed by introducing a residual negative strain ε_{0pc} furnishing the suitable compressive stress in the concrete slab prestressed separately. In conclusion, the following residual strains must be adopted in the analysis:

$\varepsilon_0 = 0$ or $\varepsilon_0 = -\varepsilon_{0pc}$ in the deck concrete and reinforcements;
$\varepsilon_0 = -\varepsilon_{0p}$ in the cable

Case B - unpropped beam prestressed after the concrete deck cast

The case of unpropped beam prestressed after the deck cast requires an initial analysis describing the behavior in the initial stage where the sole steel beam undergoes weight loads of both steel beam and concrete deck. This preventive analysis also permits evaluating the axis displacements $\bar{u}_0$ and $\bar{v}_0$ required for determining the residual strain to be considered in reinforced concrete deck for the analysis of the second stage where the whole beam (deck, steel beam and cable) is present and where prestressing can be introduced by means of an adequate value ε_{0p} of residual strain in the cable. If the concrete deck is prestressed separately before connection a further residual strain ε_{0pc} must be adopted, as already discussed in case A. In conclusion, in the analysis of the second stage the following residual strain fields have to be considered.

$\varepsilon_0 = \bar{u}_0' - y\bar{v}_0''$ or $\varepsilon_0 = \bar{u}_0' - y\bar{v}_0'' - \varepsilon_{0pc}$ in the deck concrete and reinforcements;
$\varepsilon_0 = -\varepsilon_{0p}$ in the cable

Case C - beam prestressed before the concrete deck cast

The case of unpropped beams prestressed before the concrete deck cast, differs from the previous case only because in the preventive analysis describing the structural behavior at the first construction stage, the cable is present and the prestressing force is applied by means of ε_{0p}. Once the displacement fields $\bar{u}_0$ and $\bar{v}_0$ existing in the first stage are determined, the residual strain field to be considered for the upper deck in the second construction stage is furnished by an expression formally equal to that adopted in case B, even if the axis displacements $\bar{u}_0$ and $\bar{v}_0$ are different. Separately prestressing of the concrete deck can be taken into account as in previous cases.

3. Applications

The applications consider composite beams with depth H=1870mm and with the cross-section described in detail in Fig.1.

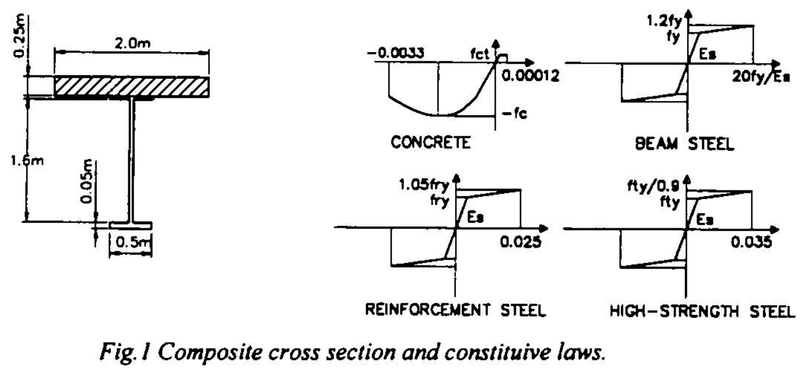

Fig. 1 Composite cross section and constitutive laws.

The concrete has a cylindrical compressive strength fc=33 MPa, tensile strength fct=2.5MPa and analyses were performed by using the constitutive law proposed in EC2 [5] (see Fig.1).Steel behavior was been modeled by means of a bilinear curve defined by elastic modulus, yield stress, failure stress and failure strain (see Fig.1). Beam and reinforcement steel have an elastic modulus Es=210 GPa while cable steel elastic modulus is Es=190 GPa. The yielding stress of beam, reinforcement and cable steel are respectively fy=280 MPa, fry=440 MPa, fty=1680 MPa.
The following applications analyze the influences of the three construction sequences by comparing the deformation and stress progress under increasing loads up to failure.

Simply supported beam

The results refer to a simply supported beam with span L=30 H, prestressed by a tendon anchored at the centroids of the end cross section and draped along a profile defined by two intermediate saddle points. In case A and B the tendon is anchored at the centroid of the composite section while in case C it is anchored at the centroid of the steel beam. The initial prestressing force is equal to 5000 kN and stress is equal to 1000 MPa in all the cases.
Fig.2 shows the load-midspan deflection curves for the three cases A, B, and C, while Fig.3 describe the maximum stress progress respectively in concrete and steel. It can be noted that the ultimate loads are almost the same in all the cases thanks to stress redistribution due to plasticity of materials while more large differences can be observed on displacements and stress distributions (Fig.2). In case B the deformation due to prestressing, applied to the complete section, approximatively balances the deformation due to dead loads, applied to the steel beam only, while in case A midspan displacements are usually positive under service loads. Case C differs from case B only because prestressing acts on the more deformable steel beam, even if large differences do not occur because the cable eccentricity is smaller in case C.

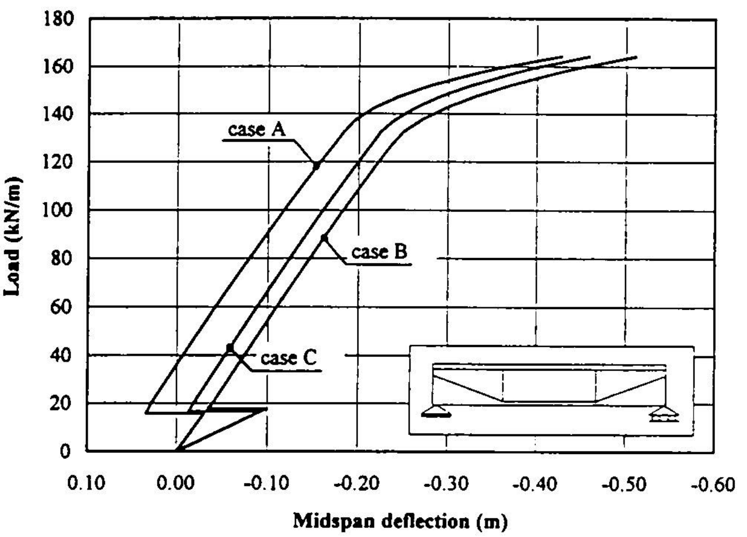

Fig.2 Midspan displacement versus load for sequences A, B, C.

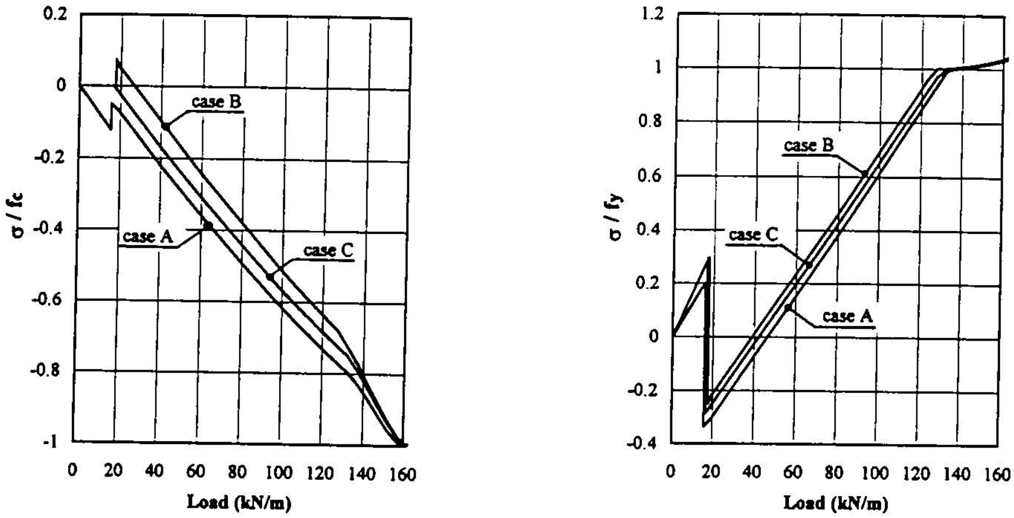

Fig.3 Progress of maximum stress in concrete (a) and steel (b).

In the three cases the stiffness progress is similar: it remains approximatively constant for a large load range and drastically reduces when the lower steel flange yields (Fig.3). In the considered cases, concrete reaches its maximum stress for a slightly larger load (Fig.3). Cable does not fail thanks to stress redistribution due to slipping even if it equally undergoes notably stress increment.
In the case A and B the prestressing force level is conditioned by upper deck cracking under dead loads while no problem exists in case C where concrete is surely compressed under dead loads, so that this latter seems to be the most effective construction sequence under the point of view of cracking prevention. Other phenomena not considered here, such as shrinkage, may contribute to increasing deck cracking.

Two-span continuous beam

A two-span continuous beam with span L=35H and a cable draped by three saddle points, one located at the intermediate support and two at the span sections with maximum moment under uniform load is considered. The lower steel flange has been enlarged in the negative moment region (thickness 120mm) so that the midspan section and support section approximatively reaches ultimate strains at the same load level. The prestressing force is equal to 7000 kN and the cable is anchored at composite cross-section centroids in case A and B and at steel beam centroids in case C. Fig.4 shows the curve load-deflection at the section where saddle points are located, Fig.5 reports the progress of concrete and steel maximum stress in the spans, Fig.6 shows the progress of maximum stress in concrete, reinforcement and the beam lower flange at the support section.

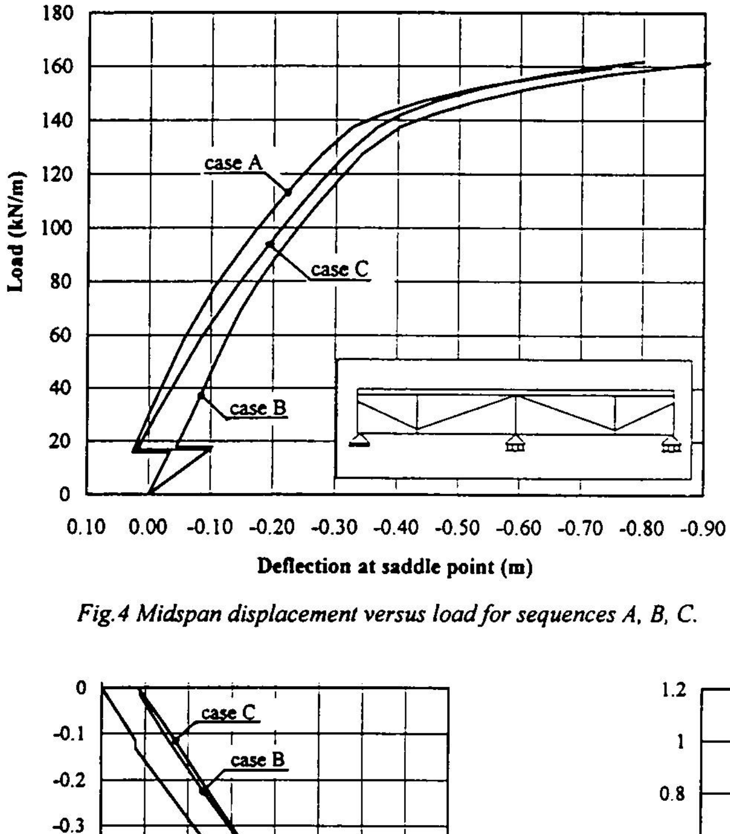

Fig.4 Midspan displacement versus load for sequences A, B, C.

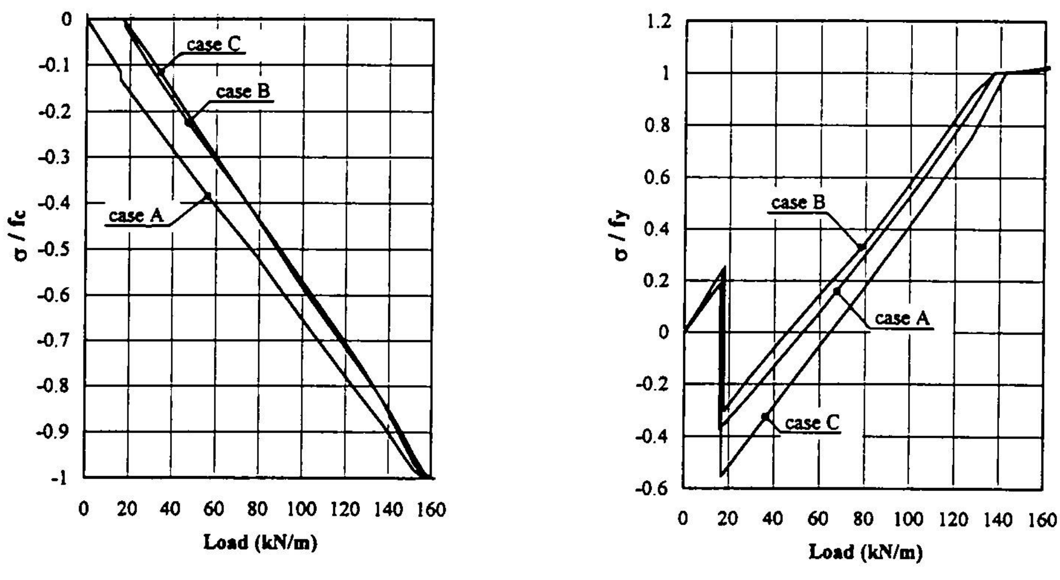

Fig.5 Midspan section: progress of maximum stress in concrete and steel.

This structural system exibhits a more complex behavior: in this case larger differences occur between displacements deriving from prestressing on the whole beam (cases A and B) and prestressing acting on the sole steel beam (case C); case C shows a lower global stiffness for a large load fields, mainly caused by concrete traction failure; steel however yields for higher loads in case C and the global stiffness achieves similar value for all the cases in the last phase. The support section attains yielding for a lower load level with respect to span section and, even in this case, plasticity of materials leads to small differences in the ultimate carrying load capacity in the three cases. The smallest deflections at low load level are observed for cases A and C while the largest deflections occur in the case B.
In the considered beam, yielding of steel precedes the concrete failure at spans (Fig.5). The stress progress in concrete are almost similar in case B and C while a higher compressive level can be observed in case A. Prestressing however has little effectiveness on concrete at span section. A noticeable reduction of tensile stress exists in the three cases and yielding occurs at higher load level in the steel beam of case C thanks to the prestressing modalities.
In continuous beams prestressing permits a sensible reduction of cracking in the region with negative moment, even if the usual prestressing advantages are partially limited by the positioning of saddle points parallel to the web that does not permit obtaining large eccentricity.
In particular, it is evident from Fig.6 that in cases A and B cracking is prevented for a large range on external loads while, contrary to the previous case of simply supported beams, no advantages can be obtained in case C. The stress progress in reinforcement rapidly increases after the deck craking in cases A and B while in case C reinforcement yields for a much lower load level (Fig.6). Prestressing shows a little effectiveness in case C while a significative reduction of compressive stress occurs in case A and B.

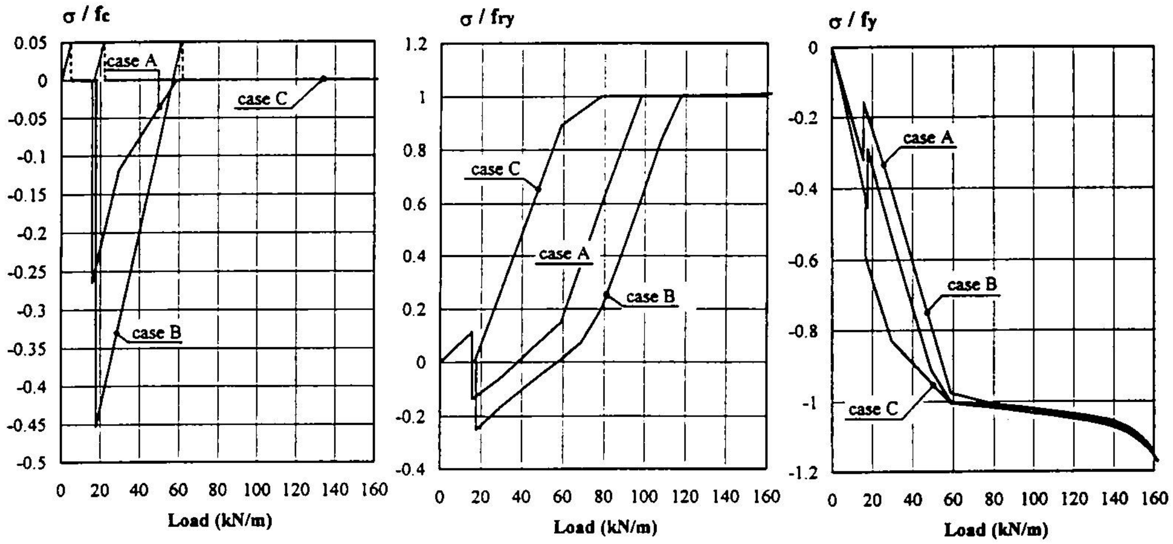

Fig.6 Section at support: progress of maximum stress in concrete, reiforcements and beam steel.

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