Consistent treatment of prestress

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Partial Prestressing with and without Bonding in Bridge Decks

Précontrainte partielle par câbles adhérents ou non dans les tabliers des ponts

Teilweise Vorspannung mit und ohne Verbund bei Fahrbahnplatten

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SUMMARY

Prestressed structures can be treated consistently with all degrees of prestressing. Basic criteria are given on how to select the appropriate solution as compared to the present practice of thinking in separate classes. This consistent approach is demonstrated for the design of bridge decks so that an optimum solution with bonded and unbonded strands can be given. It is shown that the amount of prestressed and non-prestressed reinforcement used in the slabs can be selected for various boundary conditions with consideration on aspects of reliability and economy.

RÉSUMÉ

Les structures précontraintes peuvent être traitées d'une façon cohérente à tout degré de précompression. Les critères de base permettant de sélectionner la solution optimale sont donnés, en comparaison avec la pratique actuelle des classes distinctes. Cette approche cohérente est démontrée dans le cas du dimensionnement des dalles de roulement, afin de présenter une solution optimale par câble adhérents au non. On montre ainsi que la quantité d'armature passive et précontrainte dans les dalles peut être sélectionnée pour divers conditions aux limites tout en tenant compte de la sécurité et de l'économie d'ensemble.

ZUSAMMENFASSUNG

Spannbetonkonstruktionen können einheitlich mit verschiedenen Vorspanngraden untersucht werden. Grundsätzliche Kriterien werden erläutert, wie eine geeignete Lösung zu wählen ist, im Vergleich mit dem augenblicklichen Denken in getrennten Güteklassen. Dieses einheitliche Vorgehen wird erläutert für den Entwurf von Fahrbahnplatten, damit eine optimale Lösung für Litzenspannglieder mit und ohne Verbund erreicht wird. Der Anteil an vorgespannter und schlaffer Bewehrung in den Fahrbahnplatten kann für verschiedene Randbedingungen unter Beachtung der Zuverlässigkeit und der Wirtschaftlichkeit gewählt werden.

1. Introduction

The question which appears every time when designing bridge-superstructures is: Which construction of the roadway slab in prestressed concrete has to be chosen for different boundary conditions taking into account durability and economy?

The general opinion which assumes that there is an increase of quality from reinforced concrete to partial prestressing and up to limited or even full prestressing needs to be corrected, because this simple point of view is not correct. This thinking in different quality classes must be overcome by summing up the whole range to structural concrete [1].

2. Degree of prestressing

The sign of the so far still differently named structures is the degree of prestressing κ. This degree is defined as the fraction of the whole sum of actions, which - together with the chosen prestressing - is leading to decompression at the unfavourable cross-section-fibre, this means a concrete tension zero.

Regarding beam structures under bending with axial force, this definition corresponds to the ratio of the internal forces - decompression moment to load moment -, which are related to the relevant kern point. The degree of prestressing has the following clearly defined boundaries:

$\kappa = 0$ reinforced concrete
$\kappa = 1.00$ full prestressing
$\kappa \simeq 0.70$ to 1.00 limited prestressing

The partial prestressing covers the range of $\kappa=0$ to approximately 0.70, because only for the κ-fold part of the complete actions in the service state there are compressive stresses in the whole examined cross-section.

The practical application shows that especially the degree of prestressing from 0.40 to 0.70 often results in constructively and economically favourable solutions. However, values of $\kappa \leq 0.40$ can only be used efficiently to improve the properties of reinforced concrete in the service state [2].

$$\text{degree of prestressing } \varkappa = \frac{\sigma_{c,p}}{\sigma_{c,g+q}} = \frac{M_0 + N_0 k_0}{M_{G+Q} + N_{G+Q} k_0}$$

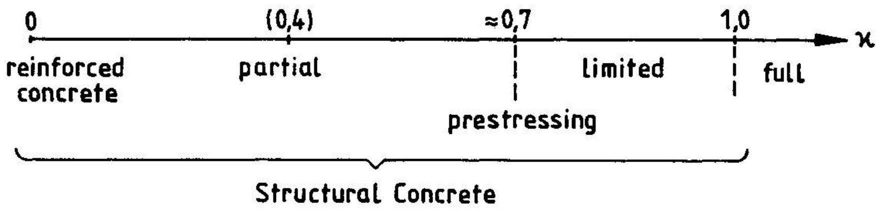

Fig.1: Definition for the degree of prestressing and the range of structural concrete

For the whole range of the structural concrete, one could give the following recommendation: Only as much prestressing as necessary, in order to achieve a favourable behaviour in the service state by means of additional axial and transverse forces with regard to deflections and reduced crack formations. But not less reinforcement as reasonable, in order to assure the durability and the reliability of the prestressed reinforced concrete concerning crack width control.

3. Restrictions of the standards

The rules of the DIN 4227, which are actually used in Germany, contain different restrictions which limit the application of the partial prestressing to a great extent. In the following they are explained by the limiting values of the concrete stresses at the unfavourable edge of the cross section:

DIN 4227, Part 1 (full and limited prestressing) prescribes that the tensile stresses - resulting of dead load, imposed deformations and 1.0-fold live load - may not exceed the given values ($\approx 2,5 \div 3.5$ N/mm^2) and that for the sum of the actions - including 0.5-fold live load - no tensile stresses appear.

In Part 2 (partial prestressing with bond) no stress checks are required, but it must be checked that for the actions including 0.5-fold live load the sheathing of the tendons is situated in the compressive area of the cross-section.

Part 6 (unbonded tendons) gives no limitations for the degree of prestressing. However, instead the general demand of the bridge authorities is relevant. This lays down that for the sum of actions and 0.3-fold live load - as the quasi permanent live load - at the unfavourable edge of the cross-section no tensile stresses might appear. This leads to the unintentional result that usually for all actions and 1.0-fold live load the tensile stresses do not exceed the values of DIN 4227, Part 1 and a planned crack formation does not occur (see the following examples).

Fig.2 shows the relation between the degree of prestressing κ and the usual ratio of live load moment to the permanent load moment, which in general is placed between 0.5 and 2.0. Moreover the required degree of prestressing, which is necessary to ensure that for the decompression moment the edge stress is zero, results with the quasi-permanent combination value ψ_Q [1] from the formula:

$$\text{decM} = M_G + \psi_Q M_Q = \kappa (M_G + M_Q)$$

This degree of prestressing results from the given simple hyperbolic formula. One can recognize three facts:

1. The transition from limited prestressing and partial prestressing does not appear at a constant value, but varies between 0.8 and 0.7 with increasing ratio of M_Q to M_G.
2. If - including 0.3-fold live load - no tensile stress is required, the degree of prestressing can only be reduced by the hatched part. This means κ between 0.7 and 0.55.
3. If one follows a proposal of Menn [3] for the normal design of roadway slabs in Suisse - prestressing only for permanent load, i.e. $\psi=0$ - the result would be a considerably greater constructive possibility. Then the degree of prestressing could be reduced to approx. 0.4.

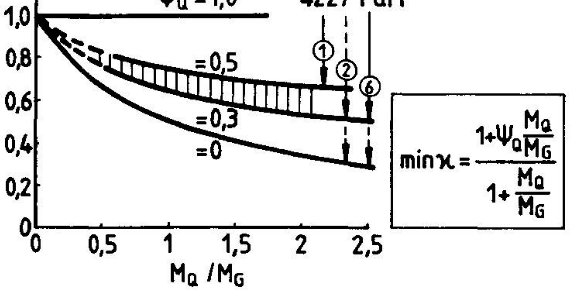

Fig.2:
Degree of prestressing for the decompression moment $M_G + \psi_Q M_Q$ with variation of ψ_Q
For bridges:
according to DIN 4227,P.1: min $\psi_Q \geq 0.5$
for Part 2+6 usually required: $\psi_Q > 0.3$

4. Application to roadwayslabs

For the construction and the design of roadway slabs one has to regard some peculiarities, such as the high percentage of live loads - $M_Q/M_G = 1 \div 2$ -, the dynamic actions and the attack of deicing salt. Therefore different answers are possible to satisfy the three principal requirements: load capacity, durability and economy.

For the problematic characteristics of the different constructions, examinations were carried out [4]. Fig.3 shows the cross-sections - box girder and double T-beam - which were half of the size of a normal German motorway (BAB). Especially, the results of middle and large dimensioned cantilevers are very expressive as a decision-making help if and how much prestressing is necessary for the transverse direction of the bridge.

The length of the cantilever were changed from 3 to 4.5 m and the depth of the connection from 0.4 to 0.65 m. The other dimensions were adapted. They do not lead to unfavourable results.

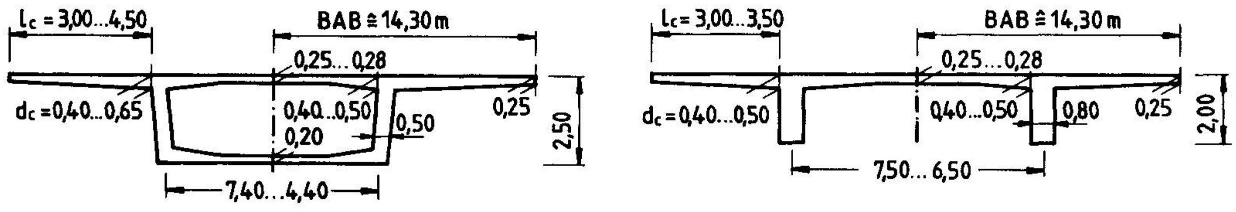

Fig.3: Form and dimension of the examined cross-sections

If the designer makes use of the partial prestressing, he has the best possibility to reach a construction with a reasonably increased amount of reinforcement and a sufficient amount of prestressing steel regarding load capacity and durability and, at the same time, a minimum of deformation.

The employment of tendons with bond requires special examinations regarding fatigue resistance and special corrosion problems - e.g. fretting corrosion and deicing salt effects.

If the internal transverse tensioning is carried out without bond, one will have the advantage of a durable corrosion protection and a larger allowable prestressing steel stress, but this construction leads to greater costs. I will leave the question beside, whether, at a later point of time, the tendons are actually changed or lengthened with widening of bridge superstructures.

Fig.4a explains the interaction between prestressing steel and reinforcing steel with different altitudes in the acceptance of the ultimate moment M_u for the concrete cantilever dimensions l_c=3.7 m and d_c=0.5 m. The sum of the A_p and the proportional A_s is shown versus the chosen degree of prestressing κ. The amount of the reinforcing steel is reduced with the ratio of the yield stresses of the reinforcing steel to the usual prestressing steel. This ratio - approx. 1:3.1 - nearly corresponds to the ratio of the costs.

You can recognize that for κ=0 and A_s=3.1·7=22 cm^2 the limit of a rational design in reinforced concrete is nearly reached. On the other hand you can see that from κ=0.5 the existing safety against rupture is greater than 1.75. If you take the additional contribution of the minimum reinforcement into consideration, the hatched saving of the prestressing steel will enlarge about this proportional amount of reinforcing steel. An economical and at the same time technical optimum is clearly situated at the partial prestressing with κ=0.5 to 0.6.

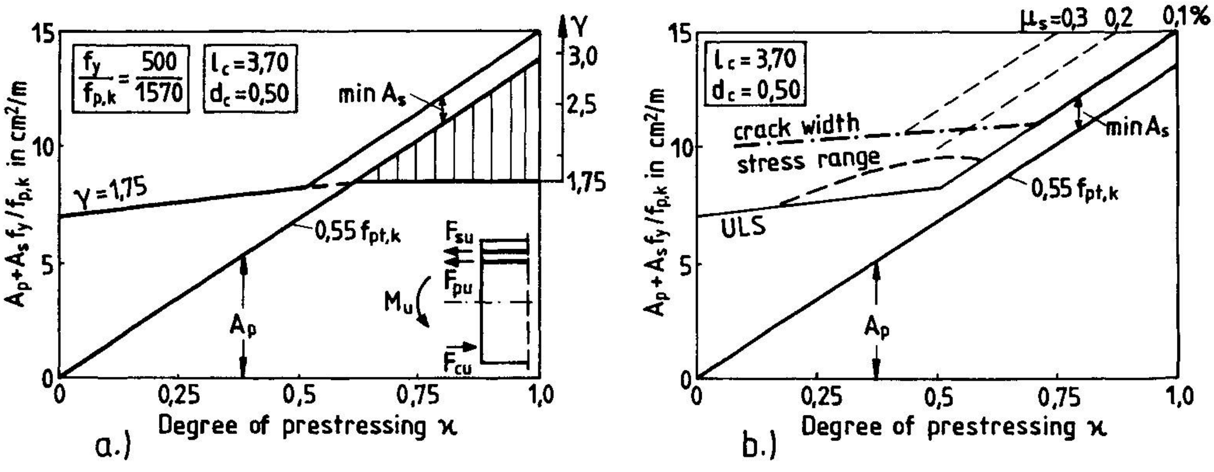

Fig.4: Proportional amount of reinforcement in a posttensioned cross-section
a.) from the check of the load capacity (ULS)
b.) from the check of the crack width and the stress range (SLS)

Among the transferred results from the ultimate limit state (ULS) the demands of the durability in the service state are supplementary analysed in Fig.4b. A substantial greater amount of reinforcing steel for the partial prestressing is the result of the crack width control according to DIN 4227 - paragraph 10.2, including ΔM - which is actually not yet conforme to DIN 1045. However, the crack width control is not responsive for limited prestressing (κ=0.75), so that you cannot make use of a reasonable percentage of reinforcement μ_s=0.2 to 0.3% for the load carrying capacity.

In the case of partial prestressing, the range of the stress amplitude of the prestressing steel might not exceed the reduced value of 110 N/mm^2 to guarantee the fatigue resistance. Moreover, Fig.4b shows that for roadway slabs this checking is not decisive and the required amount of reinforcing steel is smaller than the amount which results from the crack width control.

The demands and the knowledge which are explained in Fig.4a and 4b can be transmitted into design nomographs. These nomographs can be used as a help for the decision on the choice of the quantity and the sort of prestressing as well as for the design of cantilevers.

In the nomograph in Fig.5a you can directly see the amount of the prestressing steel which belongs to the chosen cantilever length, the cantilever depth and the requested degree of prestressing. The left dimensional line applies to $0.55f_{pt,k}$ for bonded prestressing, the right dimensional line is applicable to $0.7f_{pt,k}$ for unbonded prestressing. For the bridges which are carried out in Germany the following values result: Wannebach with κ=0.61: A_p=8.4 cm^2/m, Berbke with κ=0.66: A_p=7.4 cm^2/m.

From Fig.5b you can get the amount of reinforcing steel for the crack width control for prestressing with bond. For the example Wannebach-Bridge with l_c=4.25 m and d_c=0.62 m, you can pick out the value of 11 cm^2/m and with this the solid reinforcement of ø12, e=10cm at the cantilever connection, whereas for κ>0.75 only 5 cm^2/m would be necessary.

From the nomograph in Fig.5c you can get the tensile stresses in the uncracked state. For reasonably chosen dimensions and κ=0.5 to 0.7 the maximal stresses are not greater than the tensile stresses which are allowed for the limited prestressing. For the Wannebach- and Berbke-Bridge they reach with 2 N/mm^2 only 60% of the allowed stresses from DIN 4227, Part 1. For this reason, a planned crack formation does not appear. Simultaneously, you can directly pick out the limit of a construction in reinforced concrete with κ=0 and the maximum permissible values of transverse bending.

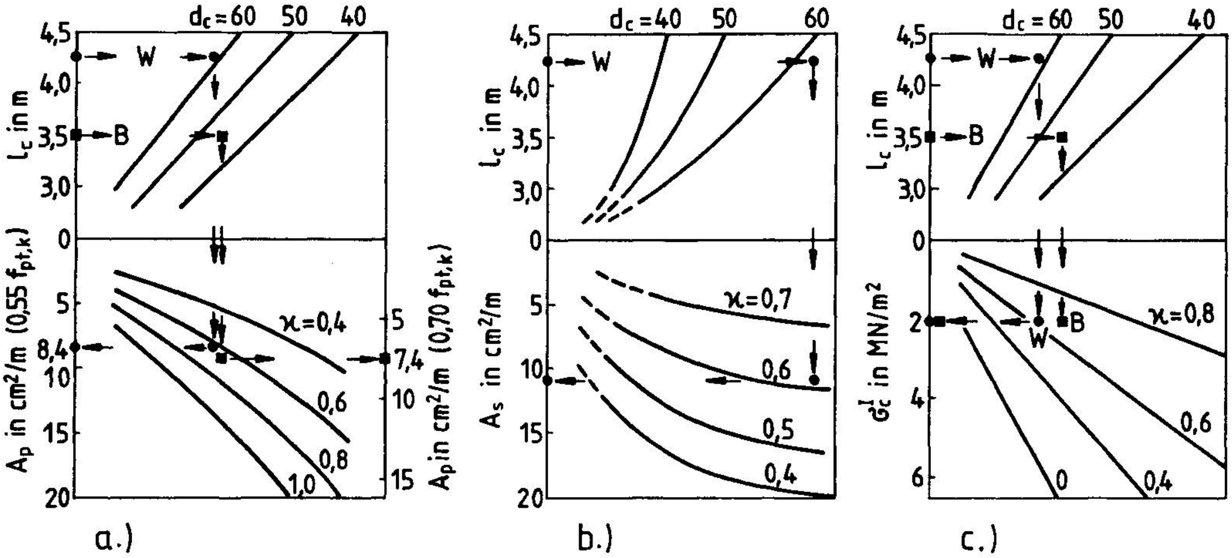

Fig.5: Nomographs for chosen cantilever dimensions (l_c and d_c) and the degree of prestressing (W=Wannebach-Bridge, B=Berbke-Bridge as examples)

a.) Amount of prestressing steel at prestressing with bond ($0.55f_{pt,k}$) and without bond ($0.70f_{pt,k}$)

b.) Amount of reinforcing steel of partial prestressing with bond resulting from the crack width control (w_{min} for ø12 mm)

c.) Edge stresses of the concrete in the uncracked state

5. Summary

With these general diagrams, which are based on the examinations of M. Empelmann [4], the designer has fundamental decision-making helps at his disposal to answer the question, which was submitted at the beginning: Whether or how much prestressing in combination with a sufficient reinforcement has to be chosen. In order to reach an economical and technical optimum, the degree of prestressing can be recommended to κ=0.5÷0.7 and the percentage of reinforcing steel to μ_s=0.2%÷0.3%. Further details about the carrried out Wannebach- and Berbke-Bridge with the consequences of the degree of prestressing can be taken from [5].

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Simple Design Method for Partially Prestressed Concrete Structures

Méthode simple de calcul pour structures en béton partiellement précontraint

Einfache Rechenmethode für den Entwurf von teilweise vorgespannten Betonkonstruktionen

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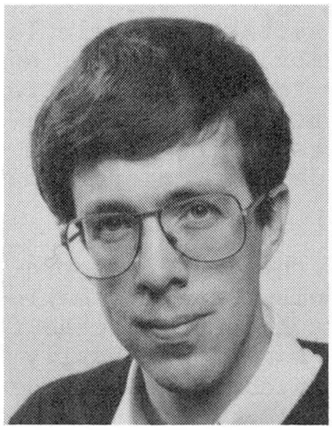

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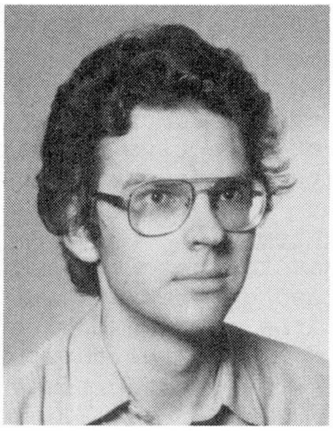

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SUMMARY

This paper presents an overview of a simple calculation method for concrete structures provided with a combination of reinforcing steel bars and post-tensioned prestressing tendons. The approach chosen relates closely to present theoretical modelling techniques aimed at a satisfactory approximation of the actual behaviour of structural concrete. The proposed method is illustrated by means of two statically indeterminate concrete structures: a rectangular girder for a warehouse and a box-girder used for a motorway.

RÉSUMÉ

Cette publication donne un aperçu d'une méthode de calcul simple pour des structures en béton pourvues d'armatures ordinaire et de câbles de post-contrainte. Cette approche a pour but d'obtenir une approximation satisfaisante pour le comportement du béton. La méthode est illustrée pour deux structures hyperstatiques en béton soit une poutre rectangulaire de magasin et un caisson de pont d'autoroute.

ZUSAMMENFASSUNG

In diesem Aufsatz wird eine einfache Rechenmethode für teilweise vorgespannten Beton vorgestellt. Die gewählte Vorgehensweise schliesst bei modernen Rechentechniken an, die zum Ziel haben, das Verhalten von Betonkonstruktionen so wirklichkeitsnah wie möglich zu beschreiben. Die vorgeschlagene Methode wird an zwei Beispielen illustriert: An einem rechteckigen Träger für ein Lagerhaus und an einer Hohlkastenbrücke für eine Autobahn.

1. INTRODUCTION

The design and behaviour of partially prestressed concrete structures have been discussed for many years [1,4,6,12,13]. However, the actual number of structural applications in The Netherlands is limited. Today, a similar situation exists in many other European countries with the exception of Switzerland [1]. Technical and economic reasons may hinder to practise research efforts, such as:

- Are cracks in concrete allowed if crossed by prestressing steel?;
- How should rather complicated calculation methods be coped with?;
- Which solution should be chosen and how does it affect the building-costs?

This paper pays attention to a rather simple calculation method applicable to crack formation in one-dimensional elements. First, the basic assumptions of the approach are briefly dealt with. Next, two structural applications are presented in chapters 3 and 4. A few conclusions are summarized in chapter 5.

2. BASIC ASSUMPTIONS

2.1 Distribution of forces

The amount of reinforcement needed in the various cross-sections is calculated on the basis of the theory of elasticity. No redistribution of forces is adopted in the ultimate limit state. The 'artificial' forces induced by prestress are either concentrated (anchorages) or distributed (pressure by curved cables). The level of effective prestressing includes losses due to friction and time-dependent material deformation which is assumed to develop unrestrained.

2.2 Cracking behaviour

The types of structural concrete may be characterized by the degree of prestressing K, defined as the ratio of the decompression moment and the maximum bending moment at the serviceability limit state [2]. Cracks are expected for $K < 1.0$ unless $\sigma_c < f_{ct}$ at the outer fibres. The cracking moment can easily be calculated if the effects of prestressing are taken into account, see figures 1a-b.

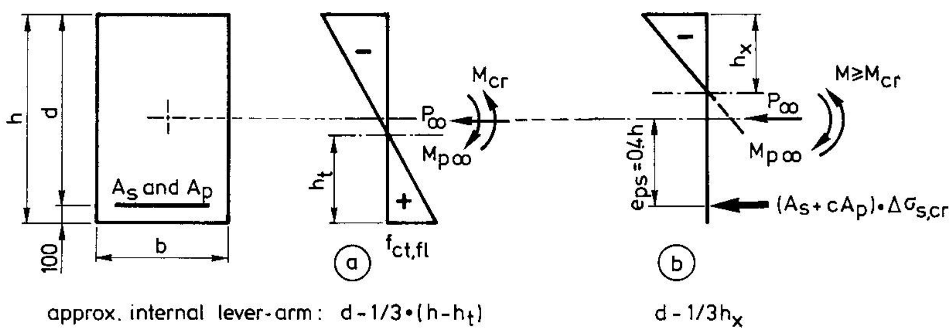

Fig. 1 Stress diagrams and internal forces (a) at and (b) after cracking.

The crack spacings and widths are found by means of theoretical models [3,4, 10,13] provided that the concrete cover is at least 1.5-2 times the largest bar diameter used. Either the pure or the flexural tensile strength is adopted as a cracking criterion for concrete. The crack width is controlled by the reinforcement. Generally, the bond stresses developed between concrete and prestressing steel are relatively low which is represented by $c < 1$:

$$\Delta\sigma_{p,cr} = c.\Delta\sigma_{s,cr} \qquad (c < 1) \qquad [MPa] \qquad (1)$$

2.3 Bending moment and shear force

The bending moment at the ultimate limit state follows from $M^* = \gamma.M_{max}$ where γ denotes the structural safety factor including material as well as loading uncertainties. A minimum amount of reinforcement is used in each cross-section (see figure 2) to ensure a distributed crack pattern and a 'tough' structural behaviour. The contribution to shear transfer due to the tendon curvature demands a sufficient axial stiffness of the tension chord [7,8]. Thus:

$$A_s.f_{sy} + A_p.f_{pk} \geq V_d \quad \text{and} \quad A_s.f_{sy} \geq V_d/2 \qquad [N] \qquad (2a\text{-}b)$$

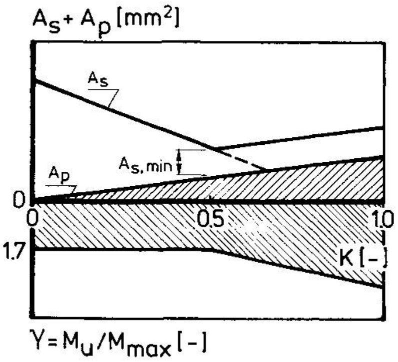

Fig. 2 Reinforcement ratio and γ as a function of the degree of prestressing.

3. STATICALLY INDETERMINATE GIRDER IN PARTIALLY PRESTRESSED CONCRETE

3.1 Introduction

The calculation method presented in chapter 2 is now illustrated by means of a continuous girder with three spans of 12.6 m each. Prefab-slabs are used supported by the beam which is part of a warehouse. The beam and the columns are monolithically connected: their bending stiffness may be neglected for the design. The dimensions of the rectangular beam are restricted to 450x1000mm²: its spacing amounts to 4.5 m. The characteristic ($\gamma = 1.0$) distributed loading amounts to: $q_p = 27$ kN/m (dead load of girder and slabs) and $q_q = 64$ kN/m (live load on slab: approx. 12 kN/m²). A safety factor $\gamma = 1.7$ is applied. The concrete cover is 50mm for the prestressing ducts. The 95% upper-bound characteristic crack width is restricted to $w_k = 0.30$mm (reinforcement) or 0.20mm (prestressing steel) [5]. Material properties: 150mm cube compressive strength $f_{cck} = 35$; $f_{sy} = 500$ and $f_{pk} = 1860$ MPa.

3.2 Reinforced concrete girder

Fourteen 25mm diameter deformed steel bars are needed in Q, i.e. $\rho_d = 1.70\%$. In The Netherlands generally a ratio of 0.8-1.2% is economic for reinforced concrete beams with a rectangular cross-section. The computations according to [5] reveal crack widths $w_k \leq 0.30$mm which agreed with an analysis based on theoretical models [10]. Vertically placed 12mm diameter closed stirrups are applied at a minimum spacing of 110mm located at cross-section Q.

3.3 Girder in fully prestressed concrete

The schematic location of the prestressing ducts is presented in figure 3. The cable was stressed at both end blocks of the beam. A fully prestressed girder could not be achieved. A good approximation was found for:

$$e_1 = e_2 = e_3 = 400\text{mm};\ R_0 = 5000\text{mm};\ R_1 = 31100\text{mm};\ R_2 = 19800\text{mm}$$

and P_0 = 2700.10³ N. Three post-tensioned elements are needed of six 12.7mm (1/2 inch) diameter strands each, thus A_p = 3x600 = 1800 mm². Due to the 'secondary moment' the line of thrust is situated 85mm above the centre line of the tendon profile at cross-section Q.

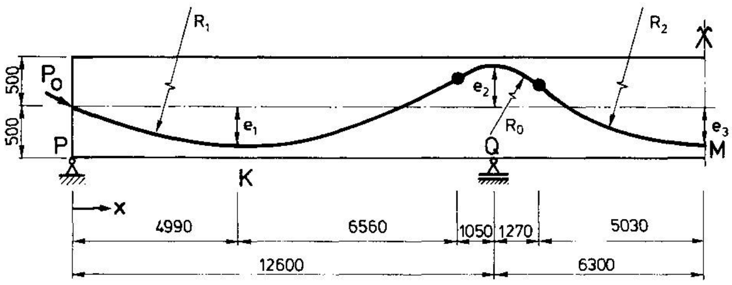

Fig. 3 Prestressing tendon profile.

3.4 Girder in partially prestressed concrete

It is proposed that no cracking may occur (or: cracks remain closed) for q_p+ $1/3q_q$ which resulted in two prestressing elements with A_p = 2x600 = 1200 mm². Additional reinforcement A_s should ensure the safety requirements. Next, the crack widths were checked: at Q, a surplus of seven 20mm diameter bars was needed. This is a reduction of 68% in comparison with the reinforced concrete girder, see also figure 4.

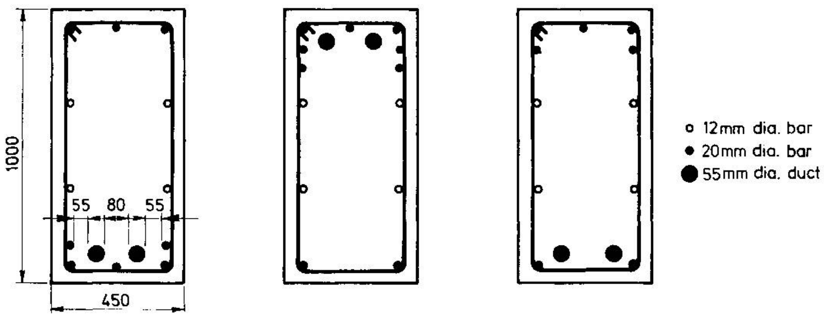

Fig. 4 Reinforcement in sections K, Q and M respectively for 0.0 < K < 1.0.

With respect to the calculated crack widths, a factor c = 0.40 was implemented in eq. (1). The empirical formula for the crack spacing presented in [2,5] was adapted to cope with prestressing effects:

$$\Delta l_m = 50 + \frac{k_2 . k_1 . d_s}{4\rho_{eff}} \qquad [mm] \qquad (3)$$

ρ_{eff} is calculated in accordance with various national codes. Eq. (3) is suitable for stabilized cracking. It followed that γ = 1.9 at K (midspan) and 1.8 at Q (support). The degree of prestressing K is at least 0.57. Vertical 12mm diameter stirrups at 300mm spacing are used throughout the structure.

3.5 Level of prestress and economy

The costs of materials (reinforcing and prestressing steel) and labour were estimated according to guide-lines presented by the Dutch building-industry (prices excl. VAT). An overview is shown in figure 5 for one girder.

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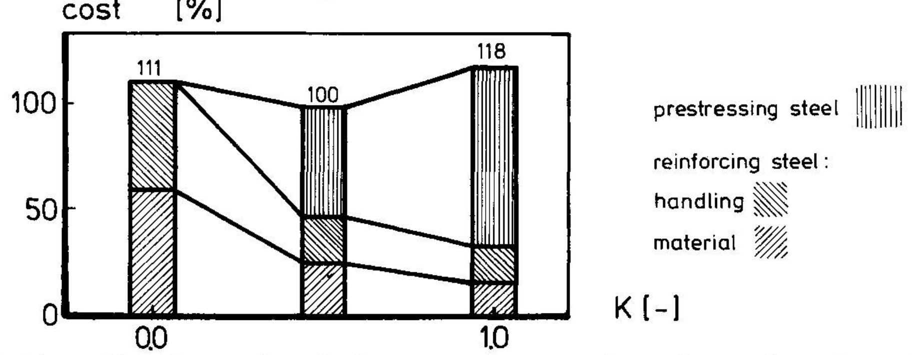

Fig. 5 Calculated distribution of reinforcement cost for three levels of prestress (100% = dfl. 5200,- = US$ 2500,- dated Oct. 1988).

4. DESIGN OF A BOX-GIRDER BRIDGE IN PARTIALLY PRESTRESSED CONCRETE

4.1 Introduction

The non-linear analysis concerns a continuous 50m span box-girder bridge with 2x2 traffic lanes subjected to dynamic traffic load, see figure 6. The design live load consists of two distributed line loads of 9 kN/m each and one 600 kN heavy-truck traffic load distributed over three axes. Load transfer of the box-girder is only considered in the longitudinal direction. Material properties: $f_{ccylk} = 36$; $f_{sy} = 400$ and $f_{pk} = 1770$ MPa.

Each of the three webs of the box-girder contains six prestressing elements of eight 15.3mm (5/8 inch) diameter strands so that $A_p = 3x6x1120 = 20160$ mm². See also figure 7. At the support and at the midspan $A_s^p = 44400$ and 33000 mm² which implies structural safety factors of 1.7 and 2.3 respectively.

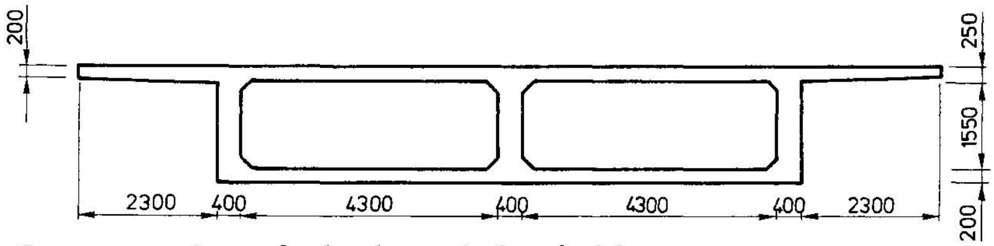

Fig. 6 Cross-section of the box-girder bridge.

Fig. 7 Tendon profile of the prestressing cables. dimensions in m

4.2 Development of cracks and steel stress variations

The average crack widths were calculated in two ways. A first approximation is based on an assumed cooperation of reinforcing and prestressing steel leading to a uniform cracking pattern. A second approach takes account of a concentrated location of the prestressing elements at the flange-web connection of the box-girder, causing a distributed cracking pattern. The average and the characteristic (95% upper-bound value) crack widths are respectively [3]:

$$w_m = \Delta l_m . \epsilon_{sm} = \Delta l_m .(\epsilon_s - \beta \Delta \epsilon_s) \quad \text{and} \quad w_k = 1.7 w_m \qquad [\text{mm}] \qquad (4)$$

where β incorporates reduced tension-stiffening ($\Delta\epsilon_s$) by a dynamic or a sustained loading. A sensitivity analysis revealed that $\beta = 0.5$ fits closely to the actual structural behaviour. The permissible crack widths (section 3.1) are not exceeded. A variation of the complete live load is related to $\Delta\sigma_p = 70$ and $\Delta\sigma_s = 130$ MPa at midspan, see also eq. (1). The permissible values ($\Delta\sigma_p = 104$ and $\Delta\sigma_s = 180$ MPa) are still empirically based.

4.3 Finite element analysis

The non-linear finite element program 'DIANA' was used in order to study the detailed structural behaviour of the girder, see in [2,9,11]. The computed longitudinal moment distribution was compared with a simple linear elastic approximation: the differences were less than 5%. The program provides a prediction of the cracking pattern at the very instant of structural failure.

5. CONCLUSIONS AND RECOMMENDATIONS

The analysis focused on partially prestressed concrete. Satisfactory results were achieved in comparison with reinforced concrete, such as: reduced crack widths and deflections, more simple detailing of the reinforcement. Moreover, the structure reacts rather insensitive to imposed deformations due to differential settlements and restraint of temperature movements or shrinkage. Applications may often be advantageous for high ratios of live to dead load or in case of a limited structural height. Partially prestressed concrete may also exhibit good economic prospects.
As stated in [4,13], extended research is needed to attain simple, consistent and reliable models which predict the behaviour of structural concrete. It may also enhance the introduction of uniform, realistic and clear design codes.

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Some Remarks on the Analytical Treatment of Prestressing

Quelques remarques au sujet du traitement analytique de la précontrainte

Einige Bemerkungen zur analytischen Behandlung der Vorspannung

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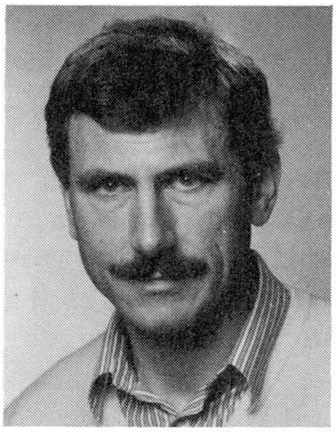


Mattias Jennewein, born 1948, studied at the University of Stuttgart, worked for five years in a consulting firm, for ten years at the Institute for Structural Design, University of Stuttgart and did there his doctorate on the design of structural concrete with strut-and-tie models.

SUMMARY
What is better: to handle prestressing as a self-strained condition or as a load? The answer to this question (shown by means of an example) is only obvious if prestressing is defined as the load which is produced by the hydraulic jack.

RÉSUMÉ
Vaut-il mieux considérer la précontrainte comme un état d'autocontrainte ou plutôt comme une charge? A l'aide d'un exemple, on montre qu'une réponse à ces questions peut être obtenue si l'on considére la précontrainte comme la charge produite par une presse hydraulique.

ZUSAMMENFASSUNG
Ist es besser, die Vorspannung als Eigenspannungszustand oder als Last zu behandeln? An einem Beispiel wird gezeigt, dass die Antworten auf alle Fragen nur einfach werden, wenn die Vorspannung als diejenige Last definiert wird, die mit der hydraulischen Presse erzeugt wird.

Some remarks on the possibilities to take "prestressing" into account in a beam with an unbonded cable (fig. 1) as an example.

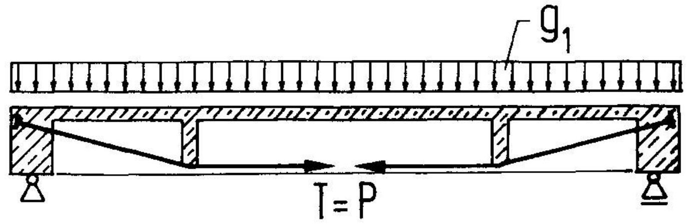


Fig. 1 Statically determinate structure during prestressing, loaded with P and g_1

During prestressing the structure is statically determinate, both internally and externally. The force T = P in the cable is determined only by prestressing. During prestressing the dead load g_1 acts as well. You can calculate all the stresses due to prestressing T = P and the load g_1. This is very simple, as you see.

However it is not simple at all if you look at it in the usual way, where prestressing is defined as the self strained condition, refering to the statically indeterminate structure after anchoring. To calculate the self strained condition, you must calculate the influence of the dead load g_1 separately in the statically indeterminate structure. You are only able to do so, if you give up the reality and if you imagine, that the load g_1 acts from the beginning (before prestressing) on the indeterminate structure. Then the force in the cable T_{g1} attributed to dead load g_1 is substracted from the real prestressing force P.

$$"P" = P - T_{g1}$$

This reduced force "P" degenerates conceptually into an imaginary parameter called "prestressing" without any practical quality. The prestressing force "P" is not a fixed value any longer. Don't tell the man at the hydraulic jack this value, if you want a correct prestressing! This parameter depends on the load g_1. If the cable will be bonded, this parameter will even change its value from one section to the other. Furthermore it depends on time. Shrinkage and creep due to the stresses of "prestressing" and even due to the dead load reduce the self strained condition, that is to say the value of the parameter "prestressing". It is common use to speak then about "losses" of prestress.

If the structure leaves the uncracked state, you get into more trouble. What's then the meaning of "prestressing" as a self strained condition? There is no meaningful explanation! The superposition or the subdivision in independant loadcases and a self strained condition is no longer possible. Therefore the question, what does happen with the moment due to the prestressing, especially with the hyperstatic part of it in an externally indeterminate structure, and what does happen with the axial force due to prestressing, cannot be answered principally. It's pretty cold comfort to show, that the answer to that question is not very important with respect to the theory of plasticity. Equilibrium is still satisfied and compatibility is taken for granted on the "beautyfull" assumption that the materials are enough ductile.

As you can see now: the definition of prestressing as a self strained condition leads you to a complicate thinking, and you finish up a blind alley. It is able to lead you astray, if you want to have an answer on a question, which cannot be answered from this point of view. It is much simpler and furthermore generally valid to take facts as facts in their natural order. Let's start all over again.

During prestressing the structure in fig. 1 is statically determinate. The force T = P in the cable is nothing but exactly the force of prestressing P in theory as well as in practice. The man at the hydraulic jack is only told this value. During prestressing the dead load g_1 acts as well. You can calculate all the stresses due to prestressing T = P and the load g_1. You see, it is not at all necessary to know the self strained condition. By anchoring the cable, the statically determinate structure changes into an internally indeterminate one. Only the events which take place <u>after</u> anchoring as e.g. loading with additional dead loads or live loads and even shrinkage and creep act on that internally indeterminate structure and must be calculated accordingly for the actual stiffnesses of concrete, reinforcing and prestressing steel.

Any changes of stresses in the prestressing steel have to be added to its stresses due to the prestress P applied by the jack. For example, the loading with the additional dead load g_2 (fig. 2):

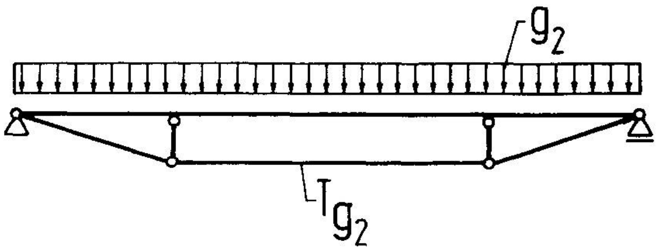


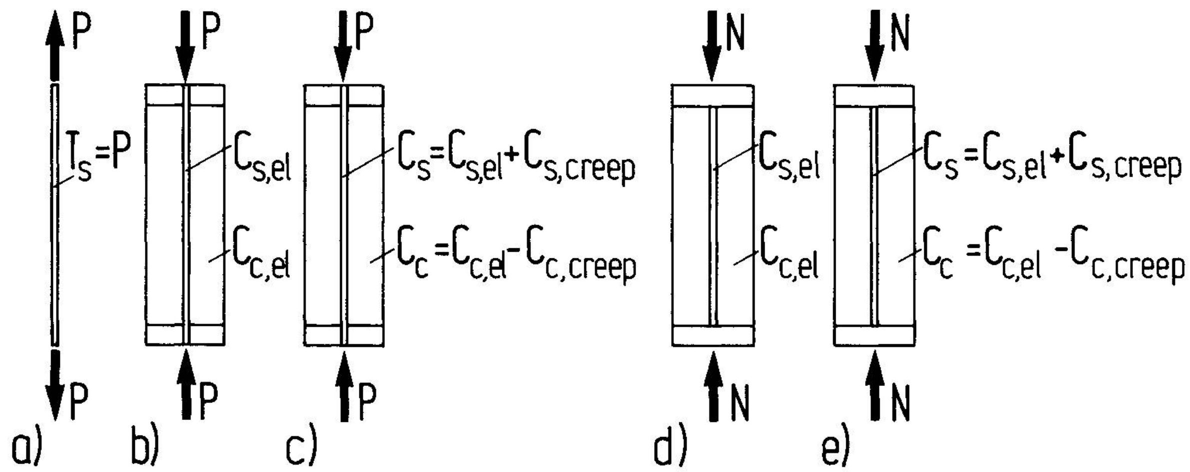
Fig. 2 Internally statically indeterminate structure loaded with g_2

The total force in the cable is now composed of the prestressing force P and of the tie force T_{g2} due to the load g_2 and amounts to

$$T_{total} = P + T_{g2}.$$

If the concrete is cracked, the actual stiffness of the beam can be taken into account. Evidently this stiffness also depends on the stresses in the beam caused by the earlier loading g_1 and P.

What about shrinkage and creep? They do not reduce the prestressing force P, which is defined as the value applied by means of the hydraulic jack once forever. There are no losses of prestress! In reality shrinkage and creep only cause a redistribution of forces between concrete and steel as it also happens in a reinforced column (fig. 3).



<u>Fig. 3</u> a) Pretensioned steel member in the prestressing bed
b) and d) same elastic behaviour of a reinforced concrete member loaded by the prestressing force P and a column loaded by the axial force N
c) and e) same behaviour of the prestressed member and the column during creep

The steel member in fig. 3a shall be pretensioned in a prestressing bed with the prestressing force P. This prestressing force P acts as an external force on the reinforced concrete member (fig. 3b), reinforced with prestressing steel (fig. 3a) which therefore works like the steel in the column (fig. 3d). The compression force $C_{c,el}$ in the concrete and the compression force $C_{s,el}$ in the steel initially depend on the elastic stiffnesses of concrete and steel (fig. 3b and 3d).

$$C_{s,el} = P * (A_s*E_s) / (A_s*E_s + A_c*E_c) = P * n*A_s/A_i$$

$$C_{c,el} = P * (A_c*E_c) / (A_s*E_s + A_c*E_c) = P * A_c/A_i$$

Respectively in the column

$$C_{s,el} = N * (A_s*E_s) / (A_s*E_s + A_c*E_c) = N * n*A_s/A_i$$

$$C_{c,el} = N * (A_c*E_c) / (A_s*E_s + A_c*E_c) = N * A_c/A_i$$

When the concrete creeps, it reduces its stress at the expense of the steel, that is to say, the part $C_{c,creep}$ of the concrete force $C_{c,el}$ is transfered to the steel as a compression force $C_{s,creep}$. The total steel force therefore amounts to

$$T_{s,total} = P - C_{s,el} - C_{s,creep}.$$

This equation preserves all events according to their occurances in reality:

- the pretensioning of the steel,
- the application of the prestressing force onto the reinforced concrete member,
- the redistribution due to creep.

In the extreme case of unlimited creep the compression force C_c in the concrete decrease to zero and the compression force in the steel grows up to $C_s = P$ in the prestressed member (fig. 3c) respectively to $C_s = N$ in the column (fig. 3e). The total steel force then is in the prestressed member

$$T_{s,total} = P - P = 0.$$

This procedure can be applied to the creep problem of the beam in the example (fig. 1). When the concrete creeps in the internally indeterminate structure, the cable force T_{creep} comes into being due to redistribution (fig. 4a).

Fig. 4b shows the statically determinate structure loaded with all its loads, that is the dead load g_1 and g_2 and the force of the cable $T_s = P + T_{g2}$. The loading, which creates creep, depends itself on the earlier created redistribution forces due to creep. For simplicity it is only shown the first short time step $\Delta t = t_1 - t_0$ with $T_{creep} = 0$ as an initial condition. In any chosen statically determinate structure creep only results in displacements and not in any force. The deformations due to creep in every section can be assumed in proportion to the elastic deformations of the concrete under the influence of the loads in fig. 4b. The resulting displacement $\delta_{10} = \delta_{creep}$ of the cable due to creep (fig. 4c) is to be reversed by the cable force X_1 (fig. 4d), which is to be determined as ΔT_{creep}.

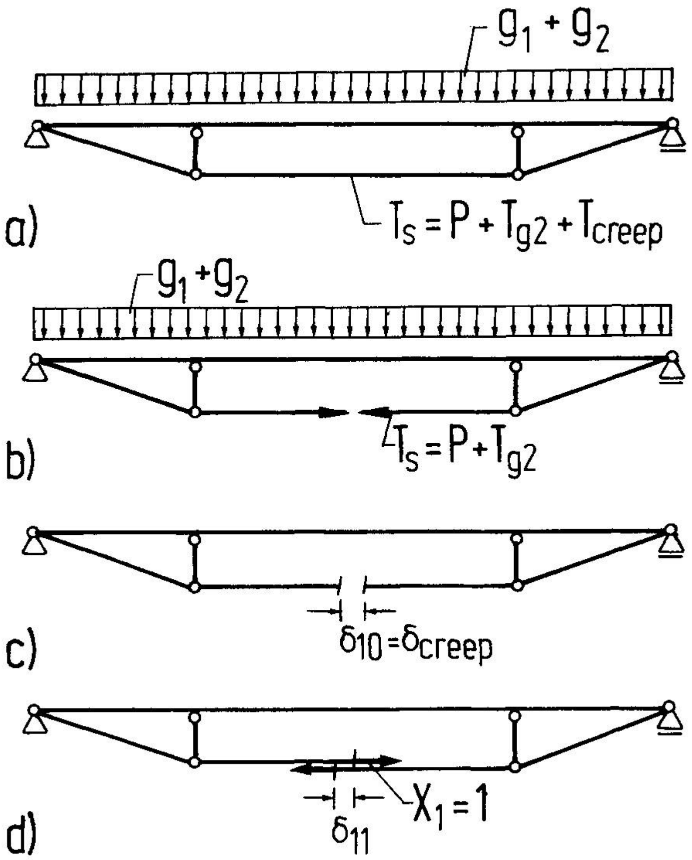


Fig. 4 a) When the concrete creeps, the tie force T_{creep} comes into being
b) the statically determinate structure and its loads
c) cable displacement $\delta_{10} = \delta_{creep}$ at the statically determinate structure, due to creep deformations
d) elastic displacement δ_{11} at the statically determinate structure due to $X_1 = 1$, considering the actual stiffness

The displacement δ_{11} of the cable due to $X_1 = 1$ depends on the actual stiffnesses, these are the stiffnesses under the influence of the loads in fig. 4b. All the internal forces of the structure due to ΔT_{creep} are redistribution forces. At the end of the first time step the total force in the cable amounts to

$$T_{total} = P + T_{g2} + \Delta T_{creep}.$$

You see, all things are kept tidy. The facts are not obscured,the relations between causes and their effects remain transparent. The procedure follows the facts and therefore it is generally valid.

Partially Prestressed Highway Bridges

Précontrainte partielle sur les ponts routiers

Teilweise vorgespannte Strassenbrücken

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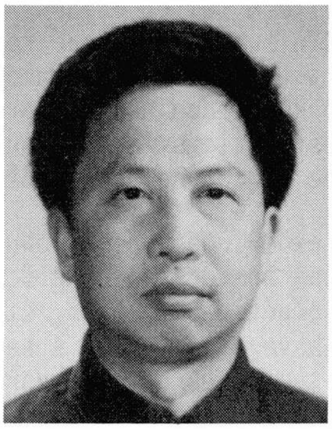

Shicheng Fan, born 1940, received his civil engineering degree from Chengdu Institute of Technology in 1960. Since 1979 he is the Head of Bridge Department of CHRI. Now, his research interests are computer application and partially prestressed concrete in bridge engineering.

SUMMARY
Systematic research on basic theories of the partially prestressed concrete bridges has been carried out. About 120 beams have been theories.This contribution presents the brief description of the experimental studies and emphasis put on some research results on flexural design. For partially prestressed concrete highway bridges, the design approach according to the prestressing degree and method to control cracking by means of the stresses of steel, are proposed.

RÉSUMÉ
Une recherche systématique sur les théories de base des ponts à précontrainte partielle a été réalisée en testant environ 120 poutres. Ce rapport présente une brève description des études expérimentales; l'importance est donnée aux résultats de recherche sur le dimensionnement à la flexion. Pour les ponts routiers à précontrainte partielle, ou propose und approche du dimensionnement basée sur le degré de précontrainte et sur le contrôle de la fissuration à travers le limitation des contraintes dans l'acier.

ZUSAMMENFASSUNG
Eine systematische Untersuchung der grundlegenden Theorien für teilweise vorgespannte Brücken wurde durchgeführt. Es wurden 120 Versuche an Balken durchgeführt. Dieser Beitrag enthält eine kurze Beschreibung der Versuche und hebt einige Forschungsergebnisse über Biegebemessung hervor. Für teilweise vorgespannte Brücken wird eine Bemessung entsprechend dem Vorspanngrad und die Rissbreitenbeschränkung über die Kontrolle der Stahl-Spannungen vorgeschlagen.

1. INTRODUCTION

The use of partial prestressing to highway bridges in China was begun in the middle seventies. At that times the design of PPC bridges was helped in the main by the existence of the European Specifications such as CEB, CP-110, FIP, as well as by informations gained from foreign previous experience and research publications. At present, while popularizing futher,the PPC at home is in the condition of that the experimental and practical engineering experience are accumulated. But up to now the PPC is getting not more generally used. Perhaps this is because of that quite a few of Chinese bridge engineers, who do not enough understand the PPC in substance, tend to be conservative and reluctant to take any risk with a new technology. Fundamentally speaking, it is accounted for the occurence that the knowledge about the strength, crack, stiffness etc. gained from tests and investigations are in sufficient, the design codes and analysis methodology on the whole are copies of the experience from abroad.

In oder to gather up more experinces on our own, to provide scientific and technical basis for revising the design code for highway bridges, to develop the design methodology fit to practice of China, since 1984, under direction of the author the CHRI and et.al have carried out a series of experimental studies and theoritical analysises on the fundamental theories of the PPC highway bridges. Based on the achieved results a more complete design recommendations and calculation system for PPC highway bridges has been proposed. In this contribution, some necessary introduction will be given in brief, but emphasis shall be merely put on the studies of crack because of the limit of the paper length.

2.BRIEF DESCRIPTION OF EXPERIMENTAL STUDIES

The whole work of the research is divided into three parts, i.e.the research on basis principles for flexural design, on basic design considerations for shear and on effects of the non-prestressing reinforcement.

The research on basic principles for flexural design of PPC beam can be summaried as follow:
(1) The ultimate flexural capacity
(2) The computing methods for normal stresses
(3) Design approach for crack control and calculation method of crack widths
(4) Calculation methods of deflection and stiffness
(5) The fatigue strength , cracking and deflection under cyclar loading
Above mentioned study tasks were brought to fruition through the rupture tests on 52 specimen beams, among them were 46 static loading and 6 cyclar loading. There were three types of tested beams. Their cross section forms involved conventional rectangular , T and I section. The forms respectively simulated the T-beams and the hollow plate beams of highway bridges. The I beams were 40cm high,prestressing by cold-strentched formed bars, but the others were 45cm high, prestressing by post-tensing high tensile strength wire tendons.The span of the specimens was 450cm long. The points of load application were symmetrically located at the one-third of span.

The studies on basic design considerations on shear includes following topics:
(1) The mechanisms of shear failure of PPC beams, the flexural capacity of inclined sections.
(2) The diagonal cracking and the computing methods for diagonal cracks.
(3) The diagonal cracking under repeated loading.
The basic data for the research on shear have been gained by the shear failure

tests on 44 beam specimens conducted in two batches. The first batch of test beams amounted to 24. Their steel percentages were just the same, but prestressing degrees, shear span ratios and stirrup percentages were distinctive. There were on 6 beams fatigue tests for inclinding sections have been accomplished.

The experimental studies on influences of non-prestresssing steel upon PPC beams were on two sides: creep of concrete and deflection. The studies have been carried out through the static tests on 20 specimen beams, among them eight have been observed over along term (above 900 days). In addition, the creep tests over 20 months on 23 concrete specimens have been made.The specimens and test beams were grouped by the grades of concrete and the ages at loading or the prestressing dergees and the steel percentages.

3. SOME RESEARCH RESULTS ON FAILURAL DESIGN

To designing the test beams the method according to prestressing degree was used.The method according to prestressing degree to design the PPC beams is a more simple, convenient, clear on idea and easy to use. Only basic checking calculations are need for designing. As the PD bring about a continuous transition from RC to PC, designing accordiing to PD is a practical and desirable unified design method for all RC, PPC and PC beams. The tests show that the tested PPC beams,designed like this (in acordance with PD), have higher ductility. The collapse of all the test beams have displaied plastic behaviour. The beam in brittle rupture have not emerged.

The measured limit flexural capacities of the tested beams of various beams having different PD, including RC beams whom PD is zero, have non-important to PD. In accordance with plasticity theory and the hypothesis of that the compressed region of concrete is a rectangle, the computing stresses are very close to the measured results. The average of the ratios of the measured stress to the computed is equal to 1.012, the standard deviation, σ=0.0645, the coefficient of the deviation, δ=0.052.

The measured deflections, strains and cracking on the varied tested beams have analogous characters. Therefore, whatever beams of RC, PPC or PC may be, a unified design approach and basic calculation methods can be used.

The strains measured on the beam specimens, prototype beams and tested bridges are better agreement with the plane section hypothesis.The average strains of concrete and steel along the depth of the tested beams is distributed as a straight line. Even to the failure moment the deformed sections all are still nearly plane.

Before cracking between the strains and loads a linear relation is kept better. After cracking, along with addition of the tested loads the increase of the strains of steel speed up,but after a short interval a relation near straight line is renewed. It is analogous to the pattern of variation in the intertia moments of the cracked sections. The stresses of concrete and steel, computing based upon elasticity theory, show very litter difference with the tested results.It can be proposed that the calculation approach like this way is dependable and exact enough.

As the modulus of rupture of concrete is not easy to define with addition of that the prestressing losses is often estimated not exactly,to estimate the cracking load of a beam accurately is not easy too. From our test results,it is can recommened that in practic the following formulae can be used to estimate the cracks moment of beams.

$Mf=(\sigma_c + R_l^b)W_o$ (1-1)

or $Mf=(\sigma_c + \gamma)W_o$ (1-2)

where σ_c = effective prestressing stress of concrete on tensile edge,
R_l^b = concrete tensile strength for designing,
W_o = resistance moment of the section to tensile edge,
γ = plastic coefficient.

Among the above two formulae the former is more conservative.

After decompressing the regular cracking patterns of PPC beams is similar to that of RC beams, thus the crack control for PPC beams can be considerated as for RC beams. The stable crack spacings of the specimens have assumed normal distribution. The aviations in the mean crack spacing are as a linear function of the d/μ or $d/\mu e$(where d--diameter of steel, μ--steel percentage, μe-steel percentage in effective region of the steel). By means of the linear regression, the mean crack spacing can be expressed as

$Lf=3.1+0.078d/\mu$ (cm) (2-1)

or $Lf=2.6c+0.18d/\mu e$(cm) (2-2)

where c-cover of outer row bars

The checking calculations show: the ratios of the maximum crack width to the mean width are always 1.4 2, the average of the ratios for the tested beams is about 1.67.

the dominant factor exerting influence on crack width is steel stress.In the service range,the variation of the crack widths with the steel stresses is linear. From the test data, it has been found that the relationship between steel stress and maximum crack width can be taken as following form:

$Wmax=a+b\sigma s$ (mm) (3)

where σs -steel stress.

This expression is tenable on varied beams, having various section forms or different PD. Based on the test data of 46 beams and used the linear regression analysis, the achieved static results are a=0.0032,b=0.599**-3. While a unit of σs is 1MPa, the correlativity coefficient R=0.8, the standard deviation, σ= 0.0652. The tests show the effect of PD upon the value of the a and the b is not distinctive. It can be seen that along with the higher PD, in a certain limit, the a trend towerds a desrease in value, but b towards a incease. The statistic results for 24 tested beams are:

a=a-0.07146(Md/Mu)**0.564 (4-1)

and b=0.6548+0.2873(Md/Mu) (4-2)

Because of the litter effect of PD on the a and b, it is reputed that the steel stresses already reflect the effect of PD. Therefore, when practic designing, to calculate the crack width the formulae (3) can be used, but the PD need not to be considerated once more. In accordance with the statistic analysis of the test data, the formula of the maximum width of crack (less than 0.3mm) is gained as following:

Wmax=0.1131+0.599**-3σ_s(mm) (5)

The guarantee percentage of this formula is 95%. Using this formula, the cracks can be controled through control to steel stresses. Based upon recent crack theories and the test data a formula for calculating maximum crack width can be easy written down as follows:

$Wmax=1.4\sigma sLf\psi/Es$ (6)

where ψ=non-uniformity factor of steel strains, to be computed from

$\psi=1.2(1-(Mf/M))$ (6-1)

Or $\psi=1.1-0.65R/(\mu e\sigma s)$ (6-2)

R-standard tensile strength of concrete,
Es-elastic modulus of steel.

Numerous checking computations show the agreement of the calculations by above mentioned formulae with test results are better.The compartions of our formulae with other formulae at home and abroad indicate that above mentioned formulae are not only reliable but also practical in designing PPC bridges.

The tests present the factitious tensile stresses of concrete bear obvious relation with the crack widths.Thus using factitious tensile stresses of concrete to control cracks is reasonable. But the tests also show:
(1) There do not exist the relationship in one by one between the factitious stresses and the crack widths.
(2) Corresponding with same factitious stresses, there may be exist large different beams.
(3) The relationships of the crack widths with the factitious stresses are different in different beams.

In recent years, using the allowable factitions tensile stresses, corresponding to the allowable crack widths,to control cracks is a usual approach in designing highway bridges. The allowable factitious stresses are stipulated in Codes, ex. JTJ 023-85(1) .The tests have discovered the allowable factitious stress in the Code JTJ 023-85 may be proper for the certain beams, but may be conservative in excess for some beams or may not on the safe side for another beams. It should be point out that a futher investigation and accumulation of experiment data must be continued. For the sake of to gain the reliable allowable factitious stresses possessed a sure guarantee percentage, the clear relatinships of factitious stresses with section forms, beam depths, prestressing type and PD must explored. The more proper calculation method for allowable factitious also must be sought.

472 measured data on 46 specimens showed both of the bilinear method and concept of effective moment of inertia (Ie) can reflect the variations in stiffness of the cracking PPC beam.

By the bilinear method the deflection of beam after cracking can be estimated from:

$$f=a **2(Mf/(a_1 Ec I_0)+(M-Mf)/(a_2 Ec I_{01})) \quad (7)$$

where I_0 and I_{01} are respectively the moment of inertia of non-cracking and cracking section. From the statistic results of the test data, the mean values of a1 and a2 are about 0.9, the standard deviation, σ=0.15, the linear correlativety coefficient R=0.95. Provided the guarantee percentage is adopted of 95%, then a1=a2=0.85, coinsided with of the Code JTJ 023-85.

A number of checking calculations indicate that, if the effective moment of inertia takes the following form:

$$Ie=I_0+(I_0-I_{01})(Mf/M)**4$$

the computed deflections agree with measured on tested beams.

The fatigue tests on the cracking PPC specimens have showed, all failures due to fatigue occurred in the non-prestresssing steel, even through the prestressed steel wires are thiner. Therefore the fatigue of PPC beam can be consideed as RC beam.the fatigue tests also have showed there is not a beam occur fatigue failure after 2 million cycles of load. If the range of cyclic stresses is simulated the stresses under the deaded loads and the maximum service loads calaculated according to the Chinese Code JTJ 021-85(2). Therefore, at the moment in designing PPC highway bridges the effect due to fatigue usually need not be considered.

4.BRIEF INTRODUCTION OF THE TRAIL BRIDGES

In order to examine the reliability of the bridges designed by use of aforecitied research results, a few of trial bridges were designed and built.There are three trial bridges tested by us. The briefs of these bridges are summaried in the following table.

Table 1. The briefs of the trail bridges

Bridge Name		Red Flag Gully		ChenjiaZhang	NandaZang
Length of bridge(m)		2*20.5+30+5*20.5		2*16	15*13
Span length of beam(m)		20.5	30	16	13
Type of section form		T	T	T	hollow plate
Beam depth(m)		175	120	110	50
Prestressing degree		0.684	0.699	0.655	0.568
prestressing steel		5×24φ5 wires		4φ25 high tensile strength formed bars	
Non-prestesssing steel		20φ14		5φ16	20φ14
Computed crack width(mm)	by CEB-FIP Model Code(3)	0.0423	0.0320	0.0462	0.0480
	by ours	0.0434	0.0379	0.0399	0.0273
Factitious tensile stess(MP)		5.75 (5.03)	5.02 (5.91)	5.83 (4.00)	4.21 (6.38)
Tensile stress in non_ prestressing steel(MPa)		68.73	54.44	64.39	49.00

note:In brackets are the allowable factitious stresses defined by Code JTJ 023_85.

These trail bridge have already been opened to traffic in succession in recent years. While constructing the Red Flag Gully Bridge the static loading test on a beam spanning 20.5m have been carried out. After put into service on the Nandazhang Bridge and the Chengjiazhuang Bridge extensive load tests under heavier vechicle loads were performed. In addition two prototype beams,which are alike of the Nandazhang Bridge, were tested. The test results prove the actual state of the beams under traffic loads is better conformable to the designed.

The success of the trial bridges led to wider recognition of the both technical and economic benifits of application of partial prestrssing in bridges.

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Recommendations on Reinforcement in Flexural and Compression Members

Recommendations concernant l'armature des structures en béton armé

Empfehlungen für die Bewehrung von Betonbauteilen

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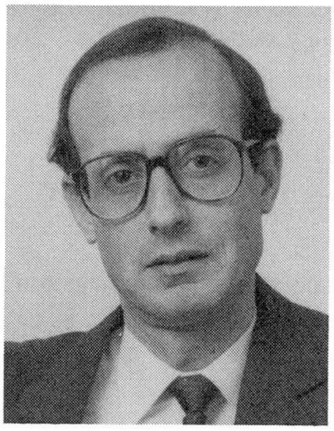

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SUMMARY

A set of recommendations related to the reinforcement in structural concrete flexural and compression members are presented. They address minimum and maximum levels of reinforcement, the percent of moment redistribution in continuous members, and the ultimate stress in the prestressing steel for bonded or unbonded tendons. The recommendations are tuned to lead to numerical results in accordance with the ACI Building Code; however, they are non-dimensional and can be applied to any code.

RÉSUMÉ

Sont présentées ici une série de recommendations concernant des structures en béton armé fléchies et comprimées. Elles concernent la qualité minimale et maximale d'armature, le pourcentage de la distribution des moments à considérer dans une structure continue, ainsi que la contrainte ultime à prendre en compte dans l'acier des câbles d'une précontrainte adhérente ou non. Les recommandations sont orientées dans le but d'obtenir des résultats numériques en accord avec le code de construction ACI; cependant, comme elles sont en fait adimensionnelles, elles peuvent s'appliquer à n'importe quel type de norme ou de code.

ZUSAMMENFASSUNG

Es werden einige Empfehlungen für die Bewehrung von Betonbauteilen unter Biege- und Normalkraftbeanspruchung dargestellt. Sie betreffen Minimal- und Maximalbewehrungsgrade, den Prozentsatz der Lastumlagerung von durchlaufenden Tragwerken und die Maximalspannung in den Spanngliedern bei Vorspannung mit oder ohne Verbund. Die Empfehlungen sollen numerische Ergebnisse in Übereinstimmung mit den ACI-Bauvorschriften geben, aber sie sind dimensionsfrei und können für jede Norm benutzt werden.

1. SCOPE

Unifying code recommendations to accommodate Structural Concrete (i.e. reinforced, prestressed, and partially prestressed concrete) in a simple and rational manner that does not violate the fundamental principles on which the provisions are based, should be an essential goal of future editions of any code of practice.

The recommendations proposed in this paper are related to the reinforcement of structural concrete members reinforced with conventional reinforcing bars, prestressing tendons, or any combination of them. The numerical values derived from these recommendations are tuned to reflect, as a reference base, the current provisions of the American Concrete Institute's Building Code Requirements (ACI 318 - 1989). However, they are written in a non-dimensionalized form and could be easily adapted to any code of practice. Some related background information can be found in [1-9].

2. FLEXURAL MEMBERS

2.1 Definition

The depth d_e from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement at nominal resistance of the section is given by the following expression (Fig. 1):

$$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y} \qquad (1)$$

where:

A_{ps}	=	Area of prestressing reinforcement in the tensile zone
f_{ps}	=	stress in the prestressing steel at nominal flexural resistance of the section (see Sections 2.5 and 5).
d_{ps}	=	distance from extreme compression fiber to centroid of prestressing steel
A_s	=	area of non-prestressed tension reinforcement
f_y	=	specified yield strength of non-prestressed tensile reinforcement
d_s	=	distance from extreme compression fiber to centroid of nonprestressed tensile reinforcement

The definition of d_e could also be easily extended to multi-layered systems, such as columns, having different layers of prestressing reinforcement and/or conventional reinforcing bars.

Note that while it is generally assumed that the reinforcing steel yields at ultimate behavior of the member, the stress, f_{ps}, in the prestressing steel is unknown and must be estimated separately (see Sections 2.5 and 5).

2.2 Maximum Reinforcement

The amount of prestressed and non-prestressed reinforcement, used for computation of moment strength of a member, shall be such that:

$$c/d_e \leq 0.42. \qquad (2)$$

where c is the depth to the neutral axis at nominal resistance in bending, and d_e is as defined in Eq. 1.

The above provision requires the determination of c, which could be obtained from writing the two equations of equilibrium of the critical section at nominal bending resistance.

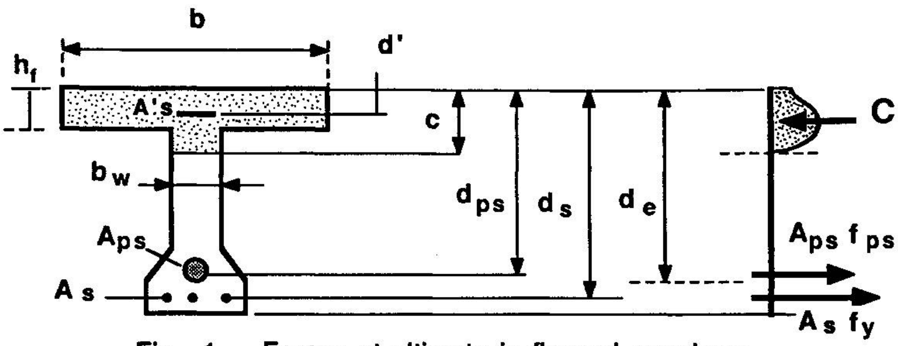

Fig. 1 Forces at ultimate in flexural members

2.3 Minimum Reinforcement

At any section of a flexural member, except where positive reinforcement is required by analysis, the amount of reinforcement shall be adequate to develop a design factored load, ϕP_n, at least 1.2 times the cracking load, P_{cr}, computed on the basis of the modulus of rupture f_r of the concrete material. Thus:

$$\phi P_n \geq 1.2 P_{cr} \qquad (3)$$

For concrete members reinforced with conventional reinforcing bars only, this provision can be satisfied by providing a minimum reinforcement ratio given by:

$$\rho_{min} \geq 0.03 f'_c/f_y \qquad (4)$$

where:

$$\rho = A_s/bd_s \qquad (5)$$

in which f'c is the compressive strength of concrete obtained from cylinder tests and other terms are as defined earlier. Note that b is taken equal b_w (Fig. 1) for T sections and joists where the web is in tension.

2.4 Moment Redistribution

Where bonded reinforcement is provided at supports in accordance with Section 18.9 of the ACI Code, negative moments calculated by elastic theory for any assumed loading arrangement may be increased or decreased by not more than

$$20(1 - 2.36 \ c/d_e) \quad \text{in percent} \qquad (6)$$

provided the value of c/d_e obtained from the design of the section at ultimate is such that:

$$c/d_e \leq 0.28 \qquad (7)$$

2.5 Stress in Prestressing Steel at Ultimate

In lieu of a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be used if f_{pe} is not less than 0.5 f_{pu}

(a) Members with bonded tendons:

$$f_{ps} = f_{pu}(1 - k \frac{c}{d_p}) \qquad (8)$$

where k is given by:

$$k = 2(1.04 - \frac{f_{py}}{f_{pu}}) \qquad (9)$$

If any compression reinforcement is taken into account when calculating f_{ps}, the value of c should be larger than or equal to 3d' to insure yielding of the compressive reinforcement. d' is defined as the depth from the extreme compression fiber to the centroid of the compressive reinforcement. If c is lesser than 3d', the contribution of the compressive reinforcement may be neglected. The basis for Eqs. 8 and 9 can be found in [1,6,7].

(b) Members with Unbonded Tendons

$$f_{ps} = f_{pe} + \Omega_u E_{ps} \varepsilon_{cu} (d_{ps}/c - 1) L_1/L_2 \leq 0.94 \; f_{py} \qquad (10)$$

where:

E_{ps} = elastic modulus of prestressing steel
ε_{cu} = assumed failure strain of concrete as per code used (i.e. 0.003 for ACI Code)
L = span length
L_1 = length of loaded span or spans affected by the same tendon
L_2 = length of tendon between anchorages
Ω_u = $3 / (L/d_{ps})$ for uniform or third point loading
Ω_u = $1.5 / (L/d_{ps})$ for one point midspan loading

In order to solve for the value of fps in Eqs (9,10), the equation of force equilibrium at ultimate is needed. Thus two equations with two unknowns (f_{ps} and c) need to be solved simultaneously to achieve a numerical solution. The background and basis for Eq. 10 can be found in [8,9].

3. COMPRESSION MEMBERS

3.1 Maximum Reinforcement in Compression Members

The areas of prestressed and nonprestressed longitudinal reinforcement for non-composite compression members shall satisfy the following two conditions simultaneously (Fig. 2):

$$\frac{A_s}{A_g} + \frac{A_{ps}}{A_g} \times \frac{f_{pu}}{f_y} \leq 0.08 \qquad (11)$$

and:

$$\frac{A_{ps} f_{pe}}{A_g \, f'_c} \leq 0.3 \qquad (12)$$

Equation 11 limits the percentage of total reinforcement in the section, while Equation 12 limits the allowable uniform compressive stress in the concrete due to prestressing, if any.

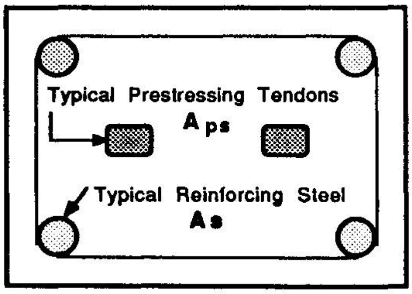

Fig. 2 Typical section of compression members

3.2 Minimum Reinforcement for Compression Members

The areas of prestressed and nonprestressed longitudinal reinforcement for non-composite compression members shall satisfy the following condition (Fig. 2):

$$\frac{A_s f_y}{A_g f'_c} + \frac{A_{ps} f_{pu}}{A_g f'_c} \geq 0.12 \qquad (13)$$

where A_g is the gross area of the compression member, f_{pu} is the ultimate strength of the prestressing tendons and other terms are as defined above.

4. PRESTRESS LOSSES - STRUCTURAL CONCRETE?

This is a subject where the general term "structural concrete" may have to be broken down into three groups, namely, reinforced, prestressed and partially prestressed concrete. Prestress losses affects only the last two groups. The accurate determination of prestress losses in prestressed and partially prestressed concrete should be based on a time step analysis. However, lump sum estimates can be used for partially prestressed as well as for fully prestressed concrete. The following remarks may be in order for partially prestressed concrete:

1. The average stress in the concrete in a partially prestressed member is generally smaller than that in a fully prestressed member. Thus the loss of prestress due to creep is also expected to be smaller.

2. If the prestressing steel is tensioned to the same initial tensile stress as in the case of fully prestressed concrete, the intrinsic relaxation loss would be the same. However, since prestress loss due to creep is smaller in a partially prestressed member, and since loss due to creep influences that due to relaxation, the relaxation loss in partially prestressed concrete members is expected to be slightly higher than in fully prestressed concrete members.

3. Everything else being equal, the loss of prestress due to shrinkage of the concrete should be the same for prestressed and partially prestressed concrete members.

4. Other instantaneous prestress losses such as friction, anchorage set, and elastic shortening can be computed in the same manner as in prestressed members.

5. The presence of a substantial amount of non-prestressed reinforcement (conventional reinforcing bars) such as in partially prestressed concrete, influences stress redistribution along the section due to creep of concrete with time, and generally leads to smaller prestress losses.

6. It is advisable to estimate creep loss on the basis of the ratio of average stress in the concrete to its compressive strength.

5. STRESS IN PRESTRESSING STEEL AT ULTIMATE - SIMPLIFIED APPROACH

In the above Section 2.5, the latest developments known to the author regarding prediction of the stress at ultimate in prestressed flexural members have been described in Eqs. 8 to 10. Such equations, combined with the equations of equilibrium at ultimate, allow for the computation of nominal bending resistance. This is as close in accuracy to a strain compatibility analysis as can be achieved to date. In an analysis or investigation situation, the combination of Eqs. 1, 8 or 10, with the two equations of force and moment equilibrium at ultimate, leads to solving four equations with four unknowns. In a design situation where, for instance, the non-prestressed steel is to be determined, an additional unknown is present. The solution becomes unnecessarily messy (involved) and its accuracy may not be needed in many design cases.

Thus, it is tempting to suggest very simplified and safe recommendations to estimate the stress at ultimate in prestressed and partially prestressed concrete. The following approach is proposed:

(a) Members with bonded tendons:

$$f_{ps} = f_{py} \quad (14)$$

This equation is always on the safe side since the limit on maximum reinforcement (Eq. 2) does not allow for the design of overreinforced members; thus actual f_{ps} will always be larger than f_{py}.

(b) Members with unbonded tendons:

$$f_{ps} = f_{pe} + 70 \quad \text{MPa} \quad (15)$$

This is generally on the safe side as observed for most of the 143 beams analyzed in [10].

Thus Eqs. 14 and 15 may be used in a first step analysis and, only if additional accuracy is needed to satisfy the design, one may revert to Eqs. 8 and 10, or to a non-linear analysis procedure.

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