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Partially Prestressed Members

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1. Introduction

In reinforced concrete structures cracks can be observed already under working loads. Through an appropriate detailing of the reinforcement the opening of the cracks is kept smaller than 0.1 to 0.3 mm. Experience over years has shown that under these circumstances corrosion of the reinforcing bars does not take place.

By prestressing a state of stress is superimposed on the stresses due to loads in such a way that no or only limited tensile stresses occur in the concrete in order to avoid cracks. Practically it is however unavoidable that secondary cracks are produced at locations where the prestressing forces are introduced or cross beams are connected to main girders. Nevertheless, they are without detrimental influence on the structural behavior.

From a theoretical point of view the principle of total prestressing, i.e. the elimination of tensile stresses in the concrete in order to eliminate all cracks, is most tempting. In its practical application difficulties are, however, often encountered. Some cases will be cited. Depending on the relative magnitude of the dead load and live load considerable variations between the minimum and maximum forces in a cross section will occur. Even greater variations will result in statically indeterminate structures. In case of full prestress, a minimum moment M_1 and a maximum moment M_2, the following four inequalities at the upper and lower fibers of a cross section must be fulfilled (Fig. 1):

Upper fiber, concrete stress:

$$\sigma_{bo} = -\frac{V}{F} - \frac{z_o}{I}(-Ve + M_1) \leq 0 \tag{1}$$

$$\sigma_{bo} = -\frac{V}{F} - \frac{z_o}{I}(-Ve + M_2) \geq \sigma_{b\,zul} \tag{2}$$

Lower fiber, concrete stress:

$$\sigma_{bu} = -\frac{V}{F} + \frac{z_u}{I}(-Ve + M_1) \geq \sigma_{b\,zul} \tag{3}$$

$$\sigma_{bu} = -\frac{V}{F} + \frac{z_u}{I}(-Ve + M_2) \leq 0\,. \tag{4}$$

The prestressing force V produces a fixed state of stress such that it is often rather difficult to compensate entirely all tensile stresses due to the moment variation $\Delta M = M_2 - M_1$. It becomes necessary to choose complicated and hence costly cross sections (e.g. box section) or apply a high prestressing force in order to satisfy conditions (1) to (4). If subsequently the condition at ultimate load is investigated the section exhibits a safety margin which is considerably above the specified one. If however condition (4) in particular is relaxed such that a limited tensile stress in the concrete is tolerated ($\sigma_{bu} \leq \sigma^*$), or condition (4) is totally neglected the choice of possible prestressing forces (i.e. choice of force V and/or eccentricity e) for a given cross sectional shape becomes greater. Instead of condition (4) a limitation of the crack opening and a sufficient safety margin against ultimate load must be imposed.

In many cases fully prestressed structures show undesirable deformations due to creep. Well known are unequal deflections of slender prestressed elements such as roof slabs. The large percentage of the live load to the total load and the high slenderness of the slab (length/thickness) cause this behavior. Even individual slabs of an identical series may show such unequal deflections.

A further situation is illustrated in Fig. 2. In an office building the column or wall loads of the upper stories are carried by a prestressed girder. If the acting live load is relatively small, the percentage of the design live load on the total load however large, a full prestress can lead to an undesirable and damaging camber of the girder. It may be continuously increased due to creep.

In statically indeterminate structures such as continuous beams full prestress may require sectional changes in the prestressing force. In the regions of intermediate supports additional prestressing cables may be necessary. They require complicated anchoring details. At such anchorage points secondary stresses occur producing very often unwanted cracks. Finally cases can be cited which exclude practically a full prestress, e.g. heavy foundation girders and slabs.

The cited examples show that the requirement of a full prestress can lead to expensive and complicated solutions. For many applications one may ask if it is necessary at all. Very often the use of a partial prestress will give a more appropriate and also more economical solution. Unnecessarily large prestressing forces will be avoided and simpler structural details can be devised. Finally even a more ductile structure may result. For full prestress would lead to such a percentage of steel that failure would occur by crushing of the compression zone without yielding of the prestressing reinforcement (brittle behavior). Especially for structures subjected to impact, earthquake or explosions such a behavior is undesirable. For their capacity of energy absorption could be deficient. In such cases partial prestress will furnish a better solution.

In the following a brief historical review of the development of prestressed concrete and the actual status of partially prestressed concrete will be given.

2. Historical Review

After unsuccessful initial attempts prestressed concrete started its development on the basic investigations of Freyssinet (patents 1928). The French school always advocated the use of a full prestress and high tension for the prestressing wires (90% of tensile strength). It is understandable that in the development phase prestressed concrete was claimed to be a completely new material (see e.g. [1] [1]), pages 35–42). A total absence of cracks was especially claimed in order to distinguish it from reinforced concrete. As late as 1951 Freyssinet was of the opinion that the use of a partial prestress was an erroneous trend worse than either prestressed or reinforced concrete.

Contrary to the development in France and many other countries Germany uses relatively low stresses for the prestressing wires (55% of tensile strength). In order to reduce the cross sectional areas of the prestressing cables and/or wires limited tensile stresses are however tolerated. The stresses are calculated on the basis of an uncracked section. The resulting tensile forces in the concrete have to be covered by an appropriate ordinary reinforcement.

The application of partial prestress was advocated even before the second world war. In particular EMPERGER [2] proposed in 1939 combined reinforcements of prestressed wires and ordinary reinforcing bars. He did not consider that cracking of reinforced concrete was a deficiency that should be avoided. The prestressed wires should be used to delay the appearance of cracks and reduce their openings.

Another method of partial prestress was proposed by ABELES in 1942 ([3], page 131) and practically applied in 1948. The total reinforcement is composed of prestressing wires and is also pretensioned. However, the prestressing force

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[1]) See list of references.

does not provide a full prestress such that under working loads tensile stresses will occur and hence also cracks with limited openings.

An international discussion on partially prestressed concrete took place for the first time at the Congress of the "Fédération Internationale de la Précontrainte" (FIP) in 1962. Subsequently a Joint Committee FIP-CEB (Fédération Internationale de la Précontrainte/Comité Européen du Béton) was created in order to study the field between reinforced concrete and prestressed concrete. Since then theoretical and experimental studies have been started in different countries. In particular a study "Enquête sur la Précontrainte Partielle" made 1965 in Belgium by R. Baus and V. Depaux should be mentioned [3]. It represents an international survey of the present status and contains pertinent references.

Up to the present the specifications of the Western Countries permit only full or limited prestress. According to assumptions no cracks should develop. Hence the stresses under working loads are calculated on the basis of a homogeneous, i.e. uncracked section. Some Eastern-European Countries allow already partial prestress for structures under predominantly static loads tolerating cracks with limited openings (e.g. DDR-Standard Spannbeton [4], page 214).

The inherent potentialities of partial prestress and the world wide interest prompted the International Association for Bridge and Structural Engineering to choose it as one of the topics for the Congress in 1968. The intention is to compile the results of the studies presently under way and to promote the application of partial prestress.

3. Classification of Partially Prestressed Structures

A number of different points of view can be taken to establish a classification of partially prestressed concrete structures. Common to all systems is the presence of controlled cracks under working loads. Specific differences show up in the technical details, especially with respect to the quality, distribution and arrangement of the reinforcement. As experimental and theoretical studies can also be classified from the same point of view this classification will also be used as a basis of discussion at the Congress.

3.1. Mixed Reinforcement

The reinforcement in a cross section consists of a prestressed and a ordinary (non-prestressed) part (Fig. 3). The prestressing wires, strands or cables are prestressed such that a chosen part of the tensile stresses in the concrete are eliminated (e.g. tensile stresses due to dead load and part of live load). The non-prestressed part of the reinforcement can be of ordinary reinforcing steel, high tensile steel or even prestressing quality. Its function is to control the cracks and deformations under working loads and to provide the required safety margin against failure. At present it appears that such a system is especially suited for post-tensioned structures.

3.2. Prestressing Reinforcement with Partial Prestress

The entire reinforcement is prestressed. Its cross sectional area is determined on the basis of the specified safety margin against failure. Under working loads tensile stresses in the concrete and also cracks can occur. Such a system is especially suited for prefabricated pretensioned elements.

3.3. Composite Beams

The beams are composed of precast prestressed elements and additional concrete parts. Fig. 4 shows two such examples. The distribution of stress over the cross section is discontinuous. The prestressed parts may act under working loads as fully or partially prestressed elements. The additional concrete parts are essentially non-prestressed (except for secondary stresses due to creep and shrinkage). On the large contact areas between the prestressed elements and the other concrete low bond stress are acting. If the contact surfaces are artificially roughened the bond will be improved and a favorable distribution and opening of cracks will result.

3.4. Combined Systems

Statically indeterminate systems such as continuous girders are sometimes composed of prestressed beams and concrete parts with ordinary reinforcement (Fig. 5). By using post-tensioned cables to establish the connection partial or even full prestress can be established. By such combined systems the interaction of parts with greatly different bending stiffnesses poses a problem, affecting the cracking behavior of the connection. Two-dimensional structures with prestress in only one direction, such as slabs, bridges with prestress in longitudinal direction etc., can also be counted under combined systems.

Another classification has been proposed by the Joint Committee FIP-CEB. Using as a criterion the tensile stresses and the corresponding cracks in the concrete the following three classes are differentiated:

Class I: The complete absence of cracks must be assured under any condition. No loading condition during construction and/or in use should produce tensile stresses. Hence theoretically at least no cracks should occur. Full prestress should be applied to structures subjected to dynamic loads producing fatigue or to a corrosive atmosphere.

Class II: Dead load and part of the live load should not produce tensile stresses. Even under extreme loading conditions the tensile strain in the concrete are limited such that in general cracks will not occur. Limited prestress can be applied to highway bridges and other structures which are seldom subjected to the full intensity of live loads.

Class III: The opening of the cracks and the strain of the reinforcement are limited. The stresses under working loads are calculated on the basis of a cracked section neglecting tensile stresses in the concrete (partial prestress).

This classification describes already the fields of application. Class III corresponds to the field of partial prestress whereas class II and class I are presently in use as limited and full prestress respectively.

It is rather difficult to determine at the present time in what direction partial prestress will develop. It appears however that a mixed reinforcement is especially suitable for post-tensioned structures. For precast pretensioned elements the use of a prestressing reinforcement, partially prestressed, may be more profitable from a practical as well as an economical point of view.

4. Principles of Design

Similar to the case of reinforced concrete structures (see e.g. CEB-Recommendations [5], p. 13) different "limiting states" can be defined:

1. Failure due to fracture of steel or crushing of concrete, instability and/or fatigue.
2. Intolerable deformations.
3. Intolerable opening of cracks.

Sufficient safety margins should be provided with respect to these limiting states. Hence it is necessary to develop reliable methods in order to determine these states in case of partial prestress.

5. Methods of Analysis

Contrary to the case of full prestress cracks in partially prestressed structures develop even under working loads. Hence stress calculations can no longer be based on the assumption of a homogeneous section. The tensile strength of the concrete must be neglected in the computations.

5.1. *Bending and Axial Load*

Firstly some remarks concerning the calculation of the cracking moment are made. Theoretically the development of the first crack is evidently given by the equation.

$$\sigma_{b\,max} = \beta_{bz} \tag{5}$$

i.e. maximum concrete stress equal to tensile strength of concrete. However, the left as well as the right side of the equation are affected by uncertainties. The tensile strength of the concrete β_{bz} is a property which can vary between wide limits. It depends on too many factors such as composition of concrete, treatment of fresh concrete, conditions during hardening, age at loading. Furthermore, cracking is influenced by the gradient of strain over the cross section and the shape of the cross section. The determination of the concrete stresses offers other uncertainties. Besides deviations in the loads unequal shrinkage and temperature distribution over the cross section, stress concentrations due to stirrups, cable conduits, etc. may greatly influence the stress distribution.

From a practical point of view the calculation of the cracking moment appears therefore questionable. A selection of allowable tensile stresses which by experience have generally not produced cracks seems more rational. In especially critical cases the limitation $\sigma_{b\,max} \leq 0$ (i.e. no tensile stress in concrete) may even be justified.

The state of stress in a cracked section coincides with the case of "bending and axial force" for reinforced concrete. Assuming a cracked cross section the calculation is made by neglecting the tensile stresses in the concrete. Fig. 6 shows a beam with mixed reinforcement, F_s being the prestressed reinforcement with a distance h_1 and F_e the ordinary reinforcement with a distance h_2 from the centroidal axis. The concrete stresses σ^b due to prestress, shrinkage and creep are σ^b_{1V} at height 1 and σ^b_{2V} at height 2 respectively. Applying two fictitious forces K_1 and K_2 the concrete is freed of all stresses (decompression of concrete). They are determined from the conditions:

$$\sigma^b_{1V} + \sigma_{11} K_1 + \sigma_{12} K_2 = 0 \tag{6}$$

$$\sigma^b_{2V} + \sigma_{21} K_1 + \sigma_{22} K_2 = 0 \tag{7}$$

where σ_{11} is the concrete stress at height 1 due to $K_1 = 1$ and the coefficients σ_{12}, σ_{21} and σ_{22} are defined correspondingly. Immediately upon prestressing or by neglecting the influence of shrinkage and creep the value of K_2 vanishes. The opposite forces $\bar{K}_1 = -\bar{K}_1$ and $K_2 = -\bar{K}_2$ together with the bending moment M and normal force N due to external loads are acting on the stress free concrete. They can be combined into an eccentrically acting resultant R:

$$R = N + \bar{K}_1 + \bar{K}_2 \tag{8}$$

$$e = \frac{1}{R}(M - \bar{K}_1 h_1 - \bar{K}_2 h_2) . \tag{9}$$

If the stress-strain relationship for the concrete, the prestressing steel and the ordinary reinforcement are known the stresses and the ultimate moment can be readily calculated assuming a linear strain variation over the cross section. The contribution of the different actions to the total strain must be taken into account. The concrete strains are only due to (R, e), hence:

$$\varepsilon_b = \varepsilon_b(R, e) . \tag{10}$$

The steel strains however, contain additional contributions due to the prestressing force V, shrinkage and creep, and the forces K_1 and K_2:

$$\varepsilon_s = \varepsilon_s(V;\ K_1, K_2;\ R, e) \tag{11}$$

$$\varepsilon_e = \varepsilon_e(V;\ K_1, K_2;\ R, e) . \tag{12}$$

In practical cases the simplest solution is often to assume as two parameters the position of the neutral axis and a strain distribution (strain in an extreme fiber). The two parameters are then varied until the stress resultants correspond to the forces due to external loads or the failure conditions are satisfied.

Besides the determination of the stresses and/or the ultimate moment information on the distribution and opening of cracks under working loads is necessary. Investigations on the cracking of reinforced concrete members (see e.g. [5], Appendix 1, page 1–93, for a list of references, furthermote [6]) may furnish useful information. The cracking behavior of partially prestressed beams is governed by the steel strains after the concrete has been freed of stresses (decompression of concrete), hence by the steel strains due to (R, e) only. Obviously the bond characteristics of the reinforcements are of prime importance. In the case of post-tensioned girders the action of the non-prestressed reinforcement is essential. For grouted cables have much inferior bond properties than deformed reinforcing bars, pretensioned wires or strands. As a first approximation the bond of the cables can be neglected for an investigation of the cracking behavior of partially prestressed beams. Design rules have to be developed in order to obtain a favorable distribution and a limited opening of the cracks. A simple limitation of the strains of the nonprestressed reinforcement constitutes only a rough design rule. A more sophisticated approach will consider all the important parameters influencing the bond behavior such as bond properties of the reinforcement, percentage of steel in the tension zone, distribution of the reinforcement, distance from the surface, quality of concrete, etc.

Until today very few systematic experimental investigations into the bending behavior of partially prestressed girders have been conducted. Besides the studies cited in [3] the tests of A. Brenneisen, F. Campus and N. M. Dehousse [7] should be mentioned. Furthermore, the AASHO Bridge Tests [8] on two pretensioned and two post-tensioned bridges and the laboratory tests of the Portland Cement Association [9] on four corresponding test girders give valuable information on the fatigue and ultimate load behavior. In order to study the fatigue behavior of partially prestressed members the working loads for half of the tests were intentionally selected to produce cracks. The partially post-tensioned girders (Bridge 5A, Girder 5A) showed a progressive deterioration of the bond and a corresponding increase in crack openings. The partially pretensioned girders (Bridge 6A, Girder 6A) however exhibited quite satisfactory behavior.

Other tests on prestressed girders conducted beyond the cracking load can be used in a similar manner in order to study the behavior of partially prestressed members.

5.2. Shear

In a partially prestressed member cracks are present even under working loads. Hence the calculation of shearing stresses assuming a homogeneous uncracked section is no longer valid. It is quite possible to compute these stresses according to the classical method for reinforced concrete beams. Assuming cracks due to bending a beam element of differential length is considered. The changes $d\sigma$ of the normal stresses σ between the left and the right section produce horizontal shearing stresses τ. It is well known that such a computation leads only to nominal values of the shearing stresses which do not correspond to the actual state. Today the concept is more and more adopted that the design of the shear reinforcement of reinforced and prestressed concrete girders should be based on the behavior at ultimate load. Numerous tests have been conducted on the shear resistance of reinforced concrete beams. However, no generally accepted shear failure theory has evolved. Nevertheless adequate and useful design rules have been recommended and even accepted into specifications. Few experimental shear investigations of prestressed concrete girders have been published. The knowledge concerning shear failure and design of shear reinforcement is correspondingly lacking.

In the field of partial prestress no systematic tests on the shear resistance have been made. In the following a recommendation is summarized for the design of the shear reinforcement of beams and slabs of reinforced, partially prestressed and prestressed concrete. It is based on a common concept for all three cases [10].

A simple description of the ultimate load behavior of beams with shear reinforcement is given by the truss analogy. In the web of the beam a system of concrete compression diagonals and tension members in direction of the shear reinforcement are developed. The two flanges may transmit beside the normal forces bending moments and shear forces depending on their state of cracking.

Fig. 7 shows a beam with a "diagonal shear" and a "bending shear" region. In the diagonal shear region only diagonal cracks in the web are present. Even at failure the tension flange remains in compression due to the prestressing force. Hence the compression flange as well as the precompressed tension flange will transmit shearing forces Q_1 and Q_2. In the bending shear region however the concrete of the tension flange is cut by a crack which continues in the web in a diagonal direction. Therefore a shear force Q_C can develop only in the compression flange. In both cases additional forces B and D can be carried by the stirrups and the diagonal reinforcement. On the basis of such a simplified model concept design recommendations have been developed in reference [10]. The shear force Q at failure must be resisted by the sum of the shear resistances:

$$Q \leq Q_C + Q_N + Q_B + Q_D \,. \tag{13}$$

The term Q_C represents the shear resistance of the concrete compression zone:

$$Q_C = \left(1 + \frac{V_\infty}{Z_s}\right) \tau_1 b_0 h \tag{14a}$$

with a maximum value

$$Q_C = 1{,}5\ \tau_1 b_0 h \,. \tag{14b}$$

Table 1: τ_1-Values

β_w (kg/cm²)	200	300	400	≥ 500
τ_1 (kg/cm²)	8	10	12	14

β_w: Cube strength of concrete after 28 days (Cylinder strength $f'_c \cong 0{,}8\,\beta_w$).

The value of τ_1 given in Table 1 depends on the compression strength of the concrete; b_0 and h are the width of the web and the effective height of the beam respectively. The influence of the prestress enters in the term V_∞/Z_s with V_∞ the prestressing force after shrinkage and creep and Z_s the yield force of the total reinforcement, i.e.

$$Z_s = F_s \sigma_{sf} + F_e \sigma_{ef} \tag{15}$$

where F_s = cross section of prestressing steel
F_e = cross section of ordinary reinforcement
σ_{sf} = yield stress of prestressing steel (0.2% permanent set)
σ_{ef} = yield stress of ordinary reinforcement (or 0.2% permanent set).

The additional resistance in the diagonal shear region is included in Q_N. This term is only introduced if the tension flange is not cracked.

$$Q_N = 0{,}2\ \sigma_N b_0 h \tag{16}$$

where σ_N is the stress in the centroide of the cross section due to the prestressing force. The resistance of the shear reinforcement is derived from the truss analogy assuming inclined cracks at 45°. For vertical stirrups

$$Q_B = \frac{F_B \sigma_{Bf} h}{t_B} \tag{17}$$

and for a reinforcement inclined at an angle α to the beam axes

$$Q_D = \frac{F_D \sigma_{Df} h}{t_D} (\sin \alpha + \cos \alpha) \tag{18}$$

with F_B, F_D = cross section of stirrups and inclined reinforcement respectively

t_B, t_D = horizontal spacing of stirrups and inclined reinforcement respectively

σ_{Bf}, σ_{Df} = yield stress (or 0.2% permanent set) of stirrup and inclined steel respectively.

No shear reinforcement is required if the nominal shear stress at failure

$$\tau = \frac{Q}{b_0 h} \leq \tau_1 . \tag{19}$$

If this value is exceeded the shear reinforcement is determined such that the inequality (13) is satisfied. The minimum reinforcement, however, should be such that

$$Q_B + Q_D \geq \tfrac{1}{2} \tau_1 b_0 h . \tag{20}$$

In order to avoid crushing of the concrete compression diagonals the following maximum shear stresses are permitted depending on the spacing of the reinforcement:

Normal spacing:

Vertical stirrups	$t_B \leq h/2$	
or	$t_B \leq 30$ cm	
Inclined reinforcement	$\alpha = 45°$	
	$t_D \leq h$	
or	$t_D \leq 40$ cm	
Maximum shear stress	$\tau \leq \tau_2 = 4\tau_1$	(21 a)

Closed spacing:

Vertical stirrups	$t_B \leq h/3$	
or	$t_B \leq 20$ cm	

Inclined reinforcement $\alpha = 45°$

$t_D \leq h/2$

or $t_D \leq 30$ cm

Maximum shear stress $\tau \leq \tau_3 = 5\,\tau_1$. (21 b)

It should be pointed out that this proposal constitutes not a new shear failure theory. It is a recommendation for a uniform and simple design procedure of the shear reinforcement of slabs and beams of reinforced, partially prestressed and prestressed concrete. The influence of the prestressing force enters into the resistance of the concrete compression zone Q_C, equation (14) and also into the additional resistance Q_N, equation (16), in case the tension zone remains precompressed. Furthermore the vertical component of the prestressing force can be deducted from the shear force if the prestressing cable is inclined with respect to the beam axis.

A comparison with tests on reinforced and prestressed beams was made in [10] and gave conservative results. For partially prestressed beams, however, a systematic experimental study is still needed to further study the design of the shear reinforcement and also the cracking behavior under working loads.

5.3. Torsion

The design of reinforced concrete members under torsion is presently studied in different countries. Similar to the case of shear the truss analogy has been used successfully in the past for the design of the reinforcement in the case of torsion. The current investigations will further clarify the situation.

The torsional behavior of prestressed members under working loads can be calculated assuming a homogeneous material if the principal tensile stresses produce no cracks. However, as in the case of bending and shear, it is not possible to predict the ultimate resistance on the basis of the elastic behavior, and hence to design the necessary reinforcement. In the case of partially prestressed members the torsional resistance at failure remains an unsolved problem.

5.4. Combined Actions

In practical applications the normal force N, bending moment M, shear force Q and torsional moment T act often in combination. Such cases occur for instance in curved bridges where torsion should be considered. The combined influence of N, M, Q and T on the failure has been recognized. However, a numerical analysis of the interaction requires generally extensive simplifications. According to most specifications for reinforced and prestressed concrete structures the actions of bending and shear are separately treated [1]). Similarly the torsional reinforcement is computed separately and simply added to the bending and shear reinforcement. It may be expected that such a procedure is conservative as long as failure is induced by yielding of the reinforcement and not by a premature crushing of the concrete (underreinforced section). Nevertheless, the situation remains unclarified. A comprehensive study of the combined actions of N, M, Q and T in girders of reinforced, partially prestressed and prestressed members would be most desirable.

6. Future Development

In the preceding sections it has been shown that partially prestressed concrete can be used already. Despite the fact that a number of questions remain insufficiently clarified the actual application should not produce unknown and unexpected problems. It has been pointed out that the control of cracking under working loads and especially the design for shear, torsion and combined actions should be investigated. The economical aspect of the application of partial prestress presents questions concerning the ratio of prestressing steel to ordinary steel, the qualities of the steel etc.

The introduction of partial prestress opens to the field of concrete structures a new dimension. It is the logical link between reinforced and prestressed concrete. How necessary such a continuous transition often is has been indicated in the introductory remarks. The designer obtains new possibilities from which he can choose the simplest and most economical solution. Under present conditions it is already cheaper to carry a large tensile force by prestressing steel than by ordinary reinforcing steel. In order to make use of the high strength of prestressing steel, however, prestressing is necessary in order to avoid large crack openings. On the other hand it is often not necessary and even undesirable to apply a high degree of prestress with the only objective to avoid tensile stresses even under extreme loading conditions. In many instances partial prestress will give the most economical solution. The prestressing steel can be essentially used to carry continuously acting loads. The additional non-prestressed reinforcement provides the necessary control of the cracks. The requirement of a full prestress in the early stage of the development of prestressed concrete was understandable. However, it has lead in many cases to complex and unnecessary expensive solutions. The technique of prestressing finds its full range of application with the introduction of partial prestress. It can be expected that it will find a much broader use than the limited field of full prestress.

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[1]) The ACI 318-63 Specifications consider the influence of the moment-shear ratio M/Qh in the shear design (Art. 1201 and 1701). As an alternative a simpler procedure is also permitted.