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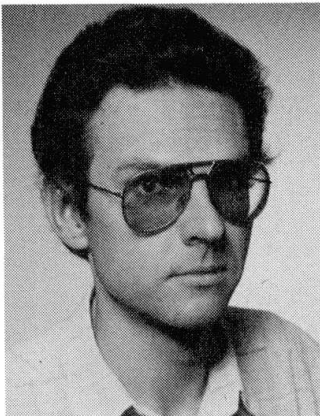
Control of Crack Width in Deep Reinforced Concrete Beams

Contrôle de la fissuration des grandes poutres en béton armé

Beschränkung der Rissbreiten in hohen Stahlbetonträgern

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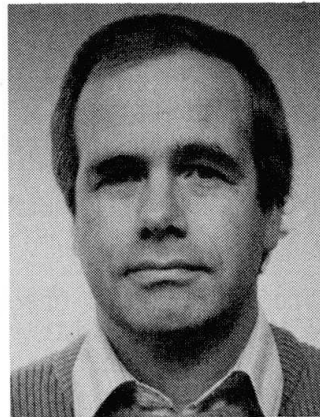
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SUMMARY

Most calculation methods for the control of crack widths in concrete structures are based on the behaviour of centrically reinforced concrete bars, subjected to tension. In deep beams, the validity of these methods is therefore mainly limited to the regions directly surrounding the main reinforcing bars. In the areas at some distance from the main reinforcement, however, crack control has to be carried out with the same attention. In this paper it is described how crack control in deep beams can be carried out on the basis of a rational model.

RÉSUMÉ

Plusieurs méthodes de calcul concernant le contrôle de la largeur des fissures sont basées sur le comportement de tirants armés axialement et tendus. Pour les murs porteurs, la validité de cette théorie est cependant limitée aux zones situées directement au voisinage des barres d'armature principale. On, pour les régions se trouvant à une certaine distance, le contrôle de la fissuration doit malgré tout être considéré avec le même soin. Ce problème est résolu de façon cohérente par un modèle rationnel adapté.

ZUSAMMENFASSUNG

Die meisten Rechenmodelle zur Beschränkung der Rissbreiten basieren auf dem Verhalten von zentrisch bewehrten Stahlbetonstäben, die auf Zug beansprucht werden. In grösseren Bauteilen ist die Gültigkeit dieser Rechenmodelle deshalb auf die Umgebung der Hauptbewehrung beschränkt. In den von der Hauptbewehrung weiter entfernten Bereichen ist eine ausreichende Untersuchung zur Vermeidung von klaffenden Rissen jedoch genau so wichtig. In diesem Aufsatz wird beschrieben wie man, aufgrund einer rationalen Modellierung, die Rissbreiten in hohen Stahlbetonträgern beschränken kann.



1. INTRODUCTION

The cracking behaviour of reinforced concrete structures has been investigated for many years. Most research was restricted to describing crack width and crack spacing in a semi-empirical manner. In recent years considerable progress has been gained with regard to the development of rational models for crack width control: Crack widths and spacings can be calculated as a function of the concrete tensile strength, bond strength, reinforcing ratio, bar size and load level. However, since laboratory experiments provide the basis for the tuning between theory and practice, most results are restricted to relatively small concrete specimens. With regard to members loaded in bending, the experiments have shown that crack widths are controlled in an effective area around the main reinforcement. However, outside this region the cracks 'collect' if too little web reinforcement is applied. Leonhardt [1] already pointed out this phenomenon in the early sixties and defined the wide cracks in the web as 'Sammelrisse'.

Since members in practice are generally considerably larger than test-beams in laboratories, this is a problem of particular importance. It is necessary to know the amount of horizontal web reinforcement that is required to control the cracking outside the 'effective area' of the main reinforcement.

2. CRACKING BEHAVIOUR OF TENSILE MEMBERS AND BEAMS

2.1 Tensile members

The first relations to predict the crack spacing and the crack width in tensile members were based on a relatively simple calculation model [2]. The basic principles of the model can be summarized as follows: At the instant of cracking, the concrete tensile force must be carried by the reinforcing steel. At a certain distance away from a crack, the so-called transfer length l_t [2], the bond stress is zero. The whole concrete section is assumed to be in uniform tension so that a new crack can occur. At increasing elongation new cracks are formed until the crack pattern is 'fully developed', e.g. all the crack spacings vary between l_t and $2l_t$. In this situation there are no parts where the concrete stress reaches the tensile strength. Thus, the mean crack spacing l_m in a fully developed crack pattern is $1.5l_t$ [2]. After the introduction of a lower-bound value, the following formula is obtained:

$$l_m = k_1 + k_2 k_3 \frac{d_s}{\rho} \quad [\text{mm}] \quad (1)$$

The mean crack width follows from the mean tensile strain ϵ_{sm} :

$$\epsilon_{sm} = \epsilon_s \left[1 - k_5 k_6 \left(\frac{\sigma_{s,cr}}{\sigma_s} \right)^2 \right] \quad [-] \quad (2)$$

where $\sigma_{s,cr}$ and σ_s are the steel stresses in a crack at the cracking load and the service load, respectively. The variation in crack widths is accounted for by the coefficient k_4 :

$$w_k = k_4 w_m \quad [\text{mm}] \quad (3)$$

The coefficients k_1 to k_6 can be tuned so as to obtain close agreement with experimental results.

2.2 Beams

Formulae (1) to (3) can also be used to predict the crack pattern at the level of the main reinforcement of beams. In the formula (1) the mean crack

spacing was found to depend on, among other factors, the reinforcing ratio ρ . In the case of a tensile member this ratio is given by $\rho = A_s/A_c$. For beams an 'effective' concrete area around the main reinforcement is defined. In most recent approaches, this area is based on the beam height h and the effective beam depth d [3,4]:

$$\rho_{\text{eff}} = \frac{A_s}{\alpha b(h-d)} \quad [-] \quad (4)$$

It is observed that the cracks initiated by the main reinforcement 'collect' outside the 'effective concrete area'. Thus, in the web of beams fewer cracks with larger widths occur, see figure 1 [5]. Several researchers have investigated the development of crack widths and spacings over the entire height of deep beams, e.g. [6,7]. However, the amount of experimental data is rather limited. Therefore, it was decided to perform experiments on deep reinforced concrete beams.

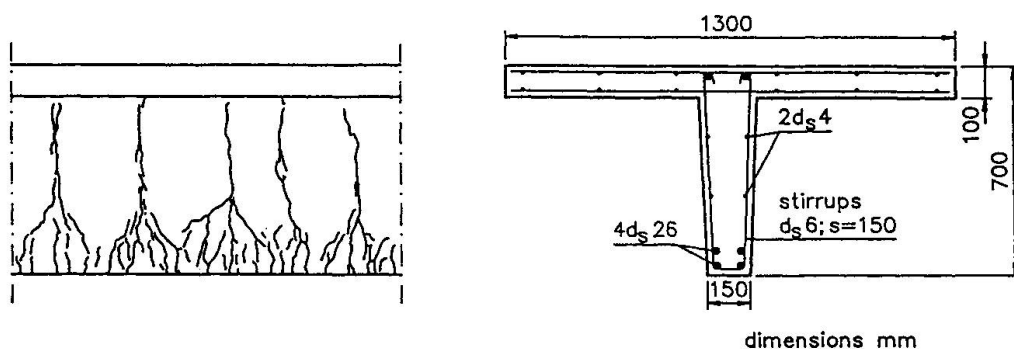


Fig. 1 Crack pattern and cross-section of a deep T-beam [5].

3. EXPERIMENTS

3.1 Test set-up

In the tests 15 beams were loaded in four-point bending, see figure 2. The beams were 5.5m long and 0.8m in height. Twelve beams had a T-shaped cross-section, whereas three beams were rectangular. For the main reinforcement either 4 bars d_{20} mm or 3 bars d_{16} mm were used. The diameter of the web rebars was 10, 12 or 16mm. The vertical bar spacing was 100, 150 or 200mm. One concrete mix was used. The average 28-day 150mm cube compressive and tensile splitting strength were 50.0 and 3.0MPa, respectively.

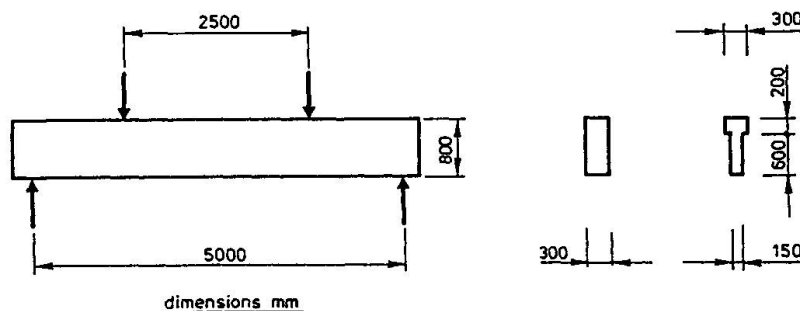


Fig. 2 Cross-sections and side-view of the beams tested



3.2 Measurements

The crack width measurements were restricted to the middle 2.3m of the uniform bending moment zone. Measurements were taken on both sides of the beams by means of a microscope. Nine horizontal lines were drawn, covering about the lower 450mm of the beams. Cracks were measured at each position where the cracks intersected these lines.

3.3 Experimental results

Figure 3 presents the influence of the web rebar spacing on the mean crack spacing. For comparison, the results of a beam without web rebars are also given. The dominant cracks are accompanied by minor cracks. Therefore, the sum of the widths of the dominant cracks is less than the measured beam elongation. This was also observed in [8]. It was found that :

$$w_m = 0.75 l_m \epsilon_{sm} \quad [\text{mm}] \quad (5)$$

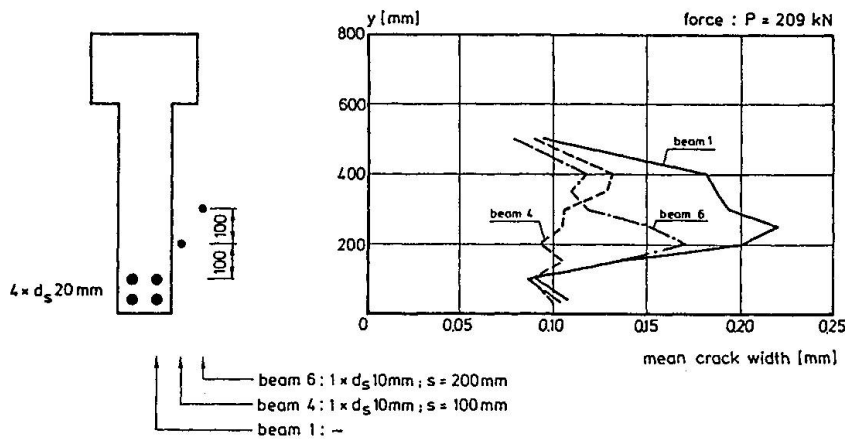


Fig. 3 The influence of web reinforcement on the mean crack width [9].

4. THEORETICAL MODEL

4.1 No web reinforcement

In the case no web reinforcement is applied, the crack pattern is similar to the one observed in a concrete wall cast on a hardened slab [10], see figure 4a. The mean crack spacing halfway down the web is approx. $l_m = h - h_x - (h - d)$. When comparing this value with the average crack spacing at the main reinforcement according to the Eurocode II a family of curves is obtained for various values of h and d (fig. 4b). The figure shows that the ratio $l_{web} / l_{main \text{ reinf.}} = 4$, which is reported by several authors, applies for the region $0.6 < \rho < 1.0\%$.

4.2 Web reinforcement

If sufficient web reinforcement is applied, cracks can be forced to extend into the web. Figure 5a presents the corresponding relation between steel stress in the web rebars, the bar diameter and the bar spacing, whereas the transfer length is shown in figure 5b. It was assumed that $f_{ct} = 2.5 \text{ MPa}$ and that the distance from the side face of the member to the centre of the web rebars is $b_1 = 50 \text{ mm}$. In the case the actual parameters differ from these values, the results from the design curves (indicated by the superscript 'd') must be corrected by the following formulae:

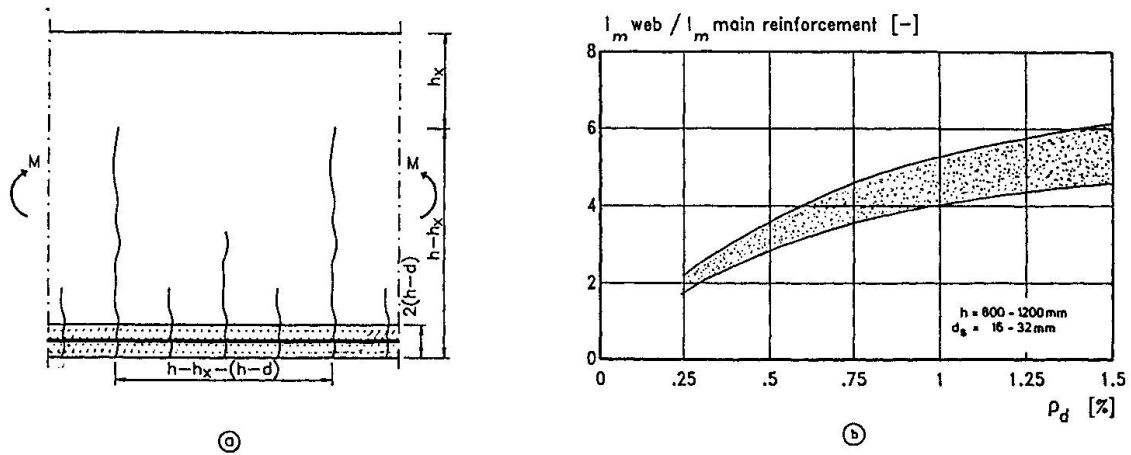


Fig. 4 Crack pattern in a deep beam without web reinforcement (a) and the ratio between the mean crack spacing in the web and at the bottom (b)

$$\sigma_{s,cr} = \sigma_{s,cr}^d (0.015b_1 + 0.125) \frac{f_{ct}}{2.5} \quad [\text{MPa}] \quad (6)$$

$$l_t = l_t^d (0.015b_1 + 0.125) \quad [\text{mm}] \quad (7)$$

The crack pattern is fully developed if the average surface strain exceeds $\sigma_{s,cr}/E_s$. The average crack spacing is then $l = l_t$. Since one aims at the use of a rather limited amount of web reinforcement, the web crack pattern is mostly not fully developed.

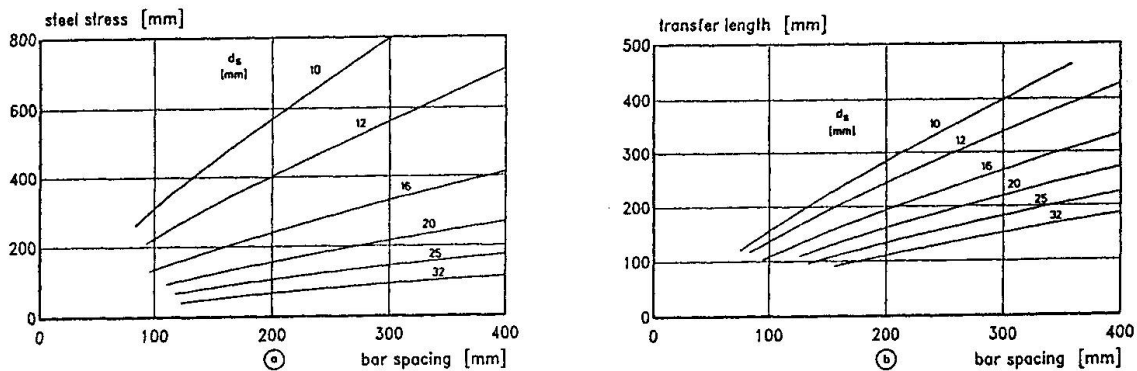


Fig. 5 The steel stress initiating cracking in the web (a) and the transfer length (b).

5. PRACTICAL DESIGN RULES

The previous sections provide the calculation method of web crack widths. Since the crack pattern is assumed not to be fully developed, long-term or varying loading is assumed to cause an increase of the transfer length by 40%. The characteristic long-term crack width is:

$$w_k = 1.3 l_t \epsilon_{s,cr} \quad [\text{mm}] \quad (8)$$



Formula (8) is presented in figure 6a in the case $f_{ct}=2.5\text{MPa}$ and $b_1=50\text{mm}$. In the case the parameters differ from the assumed values, the result from the design curve is corrected as follows:

$$w_k = w_k^d (0.015b_1 + 0.125)^2 \frac{f_{ct}}{2.5} \quad [\text{mm}] \quad (9)$$

The part of the web h_w where web reinforcement is required can be calculated according to present design rules [12].

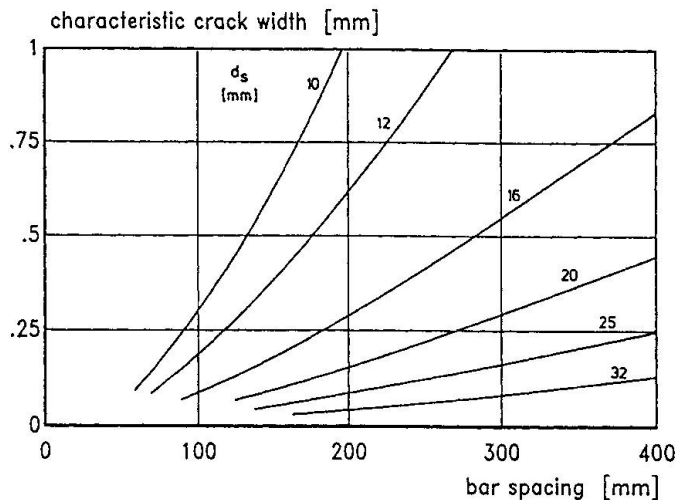


Fig. 6 Web crack width in a not fully developed crack pattern.

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