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Influence of Temperature on the Cracking in Reinforced Concrete

Influence de la température sur la fissuration du béton armé

Temperatureinfluss auf das Rissverhalten von Stahlbeton

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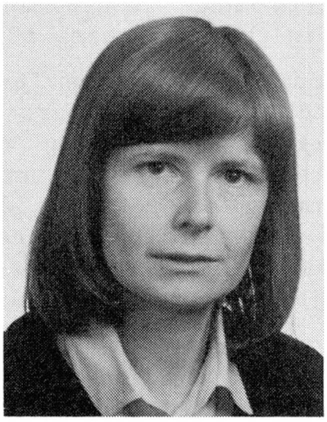

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SUMMARY

Crack spacings and crack widths in a reinforced concrete structure can be calculated by means of methods which are based on the τ-s-relation of a rebar in concrete. The τ-s-relation, presented in this paper, holds both for normal and cryogenic temperatures. From calculations it follows that the mean crack spacing at −165°C is almost the same as the value at +20°C, the mean crack width and the mean steel stress at −165°C, on the other hand are substantially greater than the corresponding values at +20°C.

RÉSUMÉ

L'espacement et l'ouverture des fissures peuvent être calculés à l'aide de méthodes basées sur la relation τ-s d'une barre d'armature dans le béton. La relation τ-s, présentée dans la présente communication, est valable pour les températures normales ainsi que pour des températures très basses. Des calculs démontrent qu'à −165°C l'espacement moyen des fissures est à peu près le même qu'à +20°C. Cependant l'ouverture moyenne des fissures ainsi que les contraintes moyennes dans l'armature sont plus élévées à −165°C qu'à +20°C.

ZUSAMMENFASSUNG

Rissabstände und Rissbreiten in einer Stahlbetonkonstruktion können mittels Methoden, die auf dem τ-s-Verhältnis eines Bewehrungsstabes im Beton beruhen, berechnet werden. Das in dieser Abhandlung beschriebene τ-s-Verhältnis gilt für normale und extrem tiefe Temperaturen. Aus Berechnungen ergibt sich, dass der mittlere Rissabstand bei −165°C ungefähr derselbe ist wie bei +20°C; die mittlere Rissbreite und die mittlere Stahlspannung sind im Gegenteil bei −165°C bedeutend grösser als die entsprechenden Werte bei +20°C.

1. INTRODUCTION

Because of the low tensile strength of concrete, both at normal and cryogenic temperatures, cracks are likely to occur in concrete constructions.

Crack control, which means the limitation of crack widths, plays a significant part in all types of structural concrete. Controlling crack widths can be applied for reasons of appearance, corrosion protection or tightness with regard to gas or liquid permeability. The cracking situation of the structural concrete has also an important influence on the extent of deformation. Deformation in cracked state can be a multiple of that in uncracked state [1,2]. The previous mentioned performance requirements are imposed by the supporting function of the structure in the serviceability limit state.

The following is an attempt to determine "quantitatively" crack spacings and crack widths in reinforced concrete members for temperatures ranging between +20°C and -165°C. The cracking behaviour of structural concrete is mainly dependent on the way a tensile force, exerted on a "reinforcement bar", is transferred to the "enveloping concrete" by way of "bond stresses". The bond between the reinforcement bar and the concrete is, consequently, one of the basic properties which make reinforced concrete possible. According to [1], concrete, reinforcing steel and bond behaviour can be called the subsystems of structural concrete.

Researchers [3,4,5,6] have found that the concrete properties, consequently also the bond stresses, are highly influenced by temperature. Especially the amount of chemically unbound water in the concrete plays an important part at low temperatures.

2. FROM τ-s RELATION TO CRACKING AT NORMAL TEMPERATURE

The bond between the reinforcement and the concrete may be described in an idealized way as a shear stress between the surface of the reinforcement bar and the surrounding concrete. The bonding mechanism may be expressed by the relation between the shear stress τ and the relative displacement s between the reinforcement bar and the concrete.

On the basis of the results of an extensive test programme of beam tests both at normal and cryogenic temperatures, executed at the Department of Civil Engineering of the K.U.Leuven, the τ-s-relation is mathematically approximated by the expression :

$$\tau = \tau_u \left(1 - \mu e^{-\lambda s}\right) \tag{1}$$

with $\mu = 0{,}78$ and $\lambda = 9{,}78$. The ultimate bond strength τ_u is a function of the concrete cover on the rebar (c), the concrete quality (f_c, f_{ct}) and the temperature :

$$\frac{c}{\phi} \leq 3 \qquad \frac{\tau_u}{f_c} = \frac{\sqrt{K}}{2}\left[1 + (1 - K)\, 0{,}353\, \frac{\frac{\phi}{2} + c}{\frac{\phi}{2}}\right] \tag{2a}$$

$$\frac{c}{\phi} > 3 \qquad \frac{\tau_u}{f_c} = \frac{\sqrt{K}}{2}\left[1 + (1 - K)\, 2{,}473\right] \tag{2b}$$

with $K = f_{ct}/f_c$. The temperature effect has been completely taken into account by way of the quantities f_c and f_{ct}. The expression (1) has the merit of describing the τ-s-course up to the bond fracture.

For a centrically loaded reinforced concrete tensile bar (Fig. 1) the transfer of the tensile force in the bar to the surrounding concrete is, for an elementary part dx, described by the following differential equation :

$$\frac{\phi \, E_s}{4(1 + \frac{E_s}{E_c}\,\omega)} \; \frac{d^2 s}{dx^2} = \tau_x \tag{3}$$

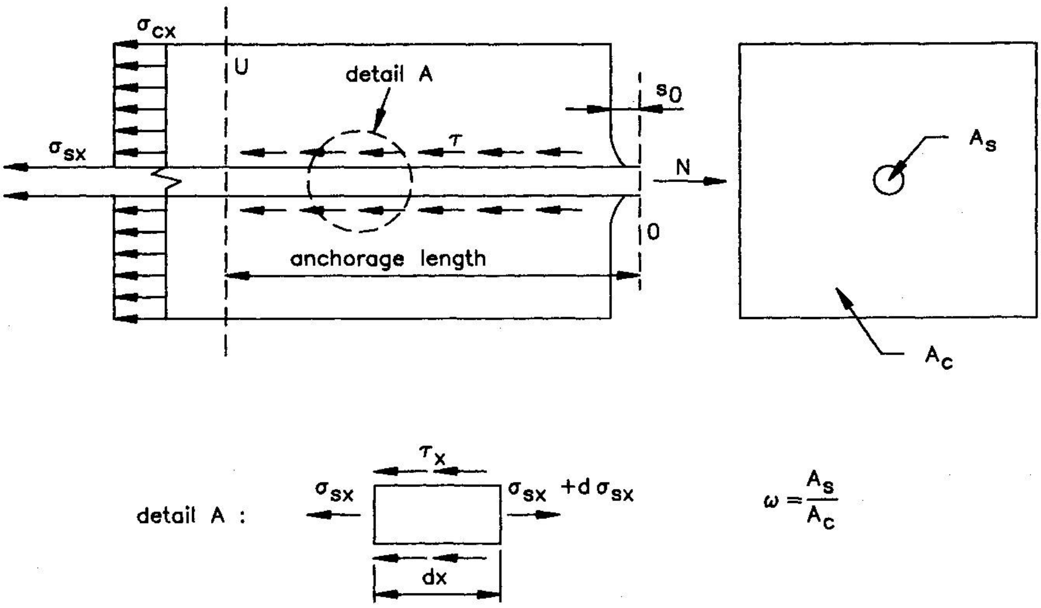


Fig. 1 Reinforced concrete tensile bar subjected to a force N.

The length, needed for the transfer of the tensile force N, is called the anchorage length. At the end of the anchorage length (in U, see Fig. 1) the concrete and the steel strain are equal. If the force N is increased in such a way that the concrete tensile stress at U becomes equal to the concrete tensile strength, i.e. $N = N_r$, the length OU is equal to the anchorage length ℓ_T. After inserting (1) in (3) and numerically solving the differential equation (3), one obtains the anchorage length ℓ_T belonging to the force N_r :

$$N_r = A_s \, \sigma_{s,r} \tag{4}$$

with

$$\sigma_{s,r} = f_{ct} \left(\frac{1}{\omega} + \frac{E_s}{E_c}\right). \tag{5}$$

When subjecting a reinforced concrete tensile bar to a force N_r it is assumed that in the first instance all "first-order cracks" are formed. A first-order crack is by definition a crack at such a distance from the nearest crack that the transfer zones of both cracks do not influence each other (Fig. 2). This requires that the distance between the two cracks in question is greater than or equal to $2\,\ell_T$. The crack width w_r, immediately after cracking is then equal to :

$$w_r = 2\, s_{r,0}.$$

After the completion of this first-order crack pattern, the so-called second-order cracks will be formed. The distance between two cracks is now smaller than $2\,\ell_T$. With the second-order cracks the transfer zones overlap partly. After the completion of the second-order crack pattern the distance between two

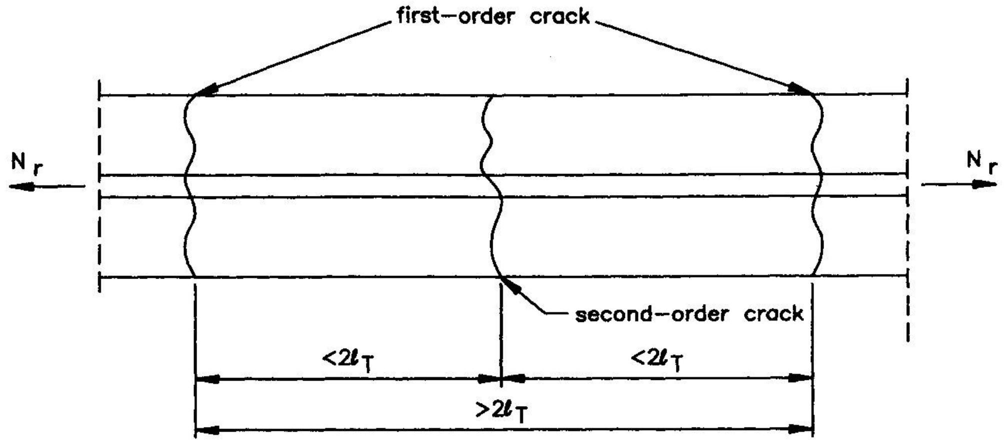


Fig. 2 First-order and second-order cracks.

cracks is at least ℓ_T. Only at this distance, taken from another crack, does the concrete stress attain the concrete tensile strength again. The crack width of second-order cracks varies, as a function of the crack spacing, between 75 % and 100 % from that of the first-order cracks.

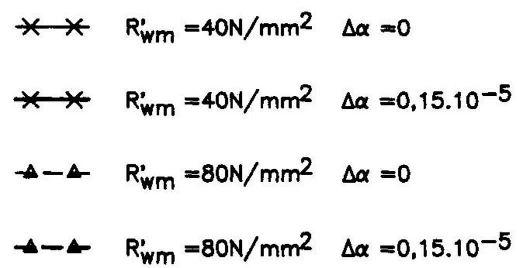
In reality the distribution of cracks is very irregular because it is determined by stochastic effects. The concrete tensile strength is, indeed, a quantity which is liable to a relatively great dispersion. Therefore strictly speaking, only minimum and maximum values can be given for the crack spacing (L_r) and crack width (w_r) respectively. A mean value for these quantities, obtained by making the calculations with the mean concrete tensile strength or another intermediate value is consequently to be interpreted with caution.

3. INFLUENCE OF TEMPERATURE ON THE CRACKING BEHAVIOUR OF STRUCTURAL CONCRETE

The above described theory may also be used for the determination of crack spacings and crack widths at low temperatures, provided that the coefficients of expansion of the concrete (α_c) and the steel (α_s) do not differ too much. This is indeed the case for concrete with a low moisture content. If, however, the concrete has a high moisture content, a correction has to be made on f_{ct}. The coefficient of expansion of the concrete is indeed at low temperatures a good deal smaller than the one of the reinforcement steel [5]. This has as a consequence that the concrete, whilst cooling off, is as it were prestressed. The tensile stresses which are formed in the steel when the temperature is going down from T_n (= normal temperature) to T_ℓ (= low temperature) may be calculated for a centrically reinforced concrete bar with :

$$\sigma_s = \int_{T_n}^{T_\ell} \frac{E_s(T)}{\left(1 + \frac{E_s(T)}{E_c(T)}\omega\right)} (\alpha_s(T) - \alpha_c(T))dT \tag{6}$$

with $E_s(T)$: modulus of elasticity of steel at temperature T (N/mm²)
$E_c(T)$: modulus of elasticity of concrete at temperture T (N/mm²)
$\alpha_s(T)$: coefficient of expansion of steel at temperature T (1/°C)
$\alpha_c(T)$: coefficient of expansion of concrete at temperature T (1/°C).



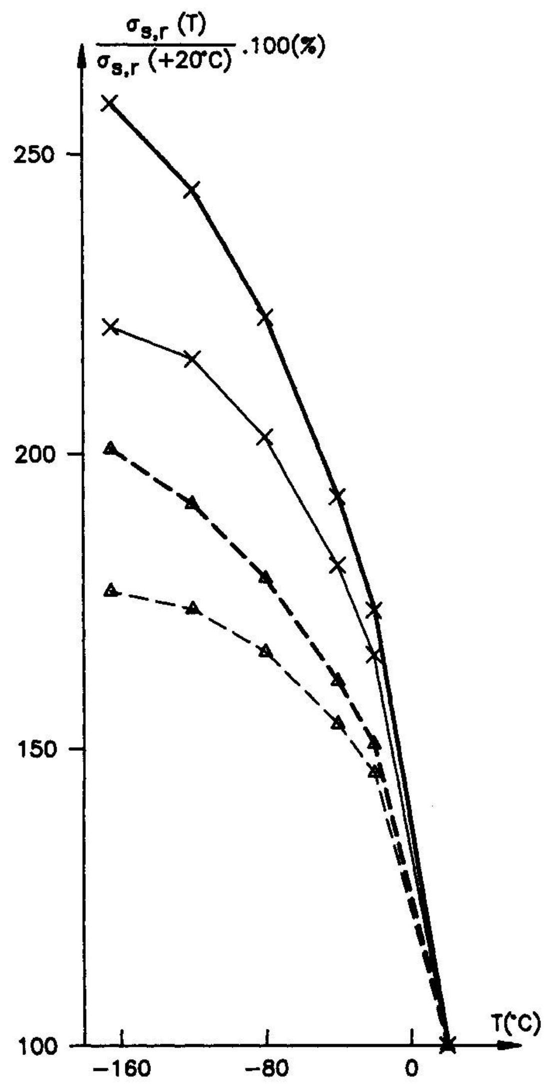


Fig. 3 Influence of temperature on the mean steel stress at the place of the crack ($\sigma_{s,r}$).

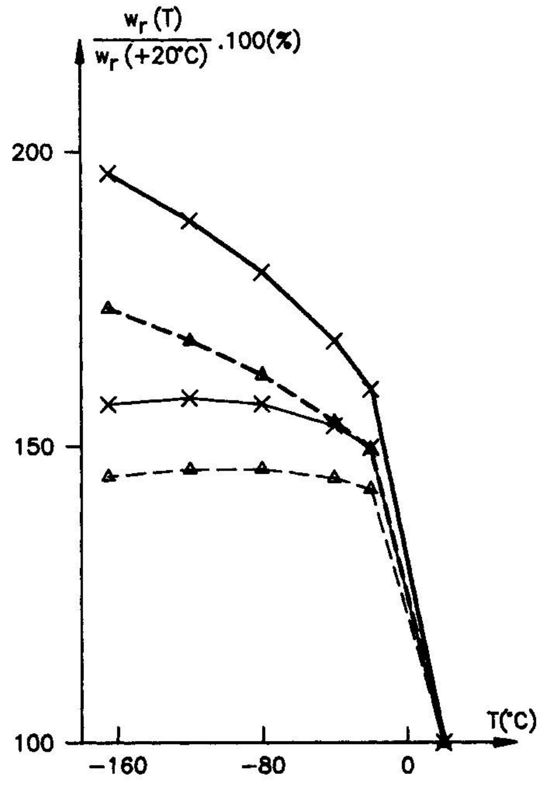


Fig. 4 Influence of temperature on the mean crack width (w_r) of first-order cracks.

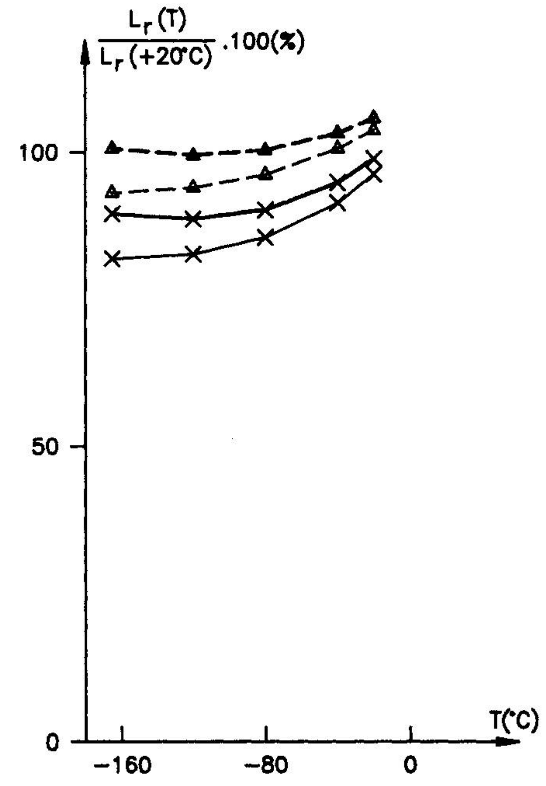


Fig. 5 Influence of temperature on the mean crack spacing (L_r) of first-order cracks.

The compressive stresses which are created in the concrete at the same time are then :

$$\sigma_c = -\sigma_s \omega . \tag{7}$$

The internal prestress may be considered a virtual increase of the tensile strength of the concrete at that temperature and thus brings about a "postponement" of the crack formation at low temperatures. In the case of concrete with a high moisture content the uniaxial tensile strength f_{ct} has to be transformed into a virtual tensile strength $f_{ct}^*(T_\ell)$ for the calculation of $\sigma_{s,r}$ by using (6) and (7) :

$$f_{ct}^*(T_\ell) = f_{ct}(T_\ell) + \omega \int_{T_n}^{T_\ell} \frac{E_s(T)}{(1 + \frac{E_s(T)}{E_c(T)} \omega)} (\alpha_s(T) - \alpha_c(T))dT . \tag{8}$$

The remaining characteristics of the concrete and the steel (f_c, f_{ct}, E_c and E_s) at low temperatures may be calculated on the basis of the corresponding values at room temperature [3,4].

Results of the qualitative course of the mean stress in the rebar at the place of the crack ($\sigma_{s,r}$), the mean crack width (w_r) and the mean crack spacing (L_r) as functions of temperature are shown in the Figs. 3, 4 and 5. At the calculation of the concrete tensile strength it is assumed that for one thing there is no difference between the coefficient of expansion of the steel and the concrete, consequently $\Delta\alpha = 0$, and for another that difference is constant over the whole temperature range and equal to $0{,}15.10^{-5}$ per °C [7].

From the Figs. 3, 4 and 5 it follows that the mean steel stress ($\sigma_{s,r}$) and the mean crack width (w_r) increase considerably, if temperature falls. This increase is so much the greater as the difference in the coefficient of expansion between steel and concrete increases and the mean compressive strength of concrete at room temperature decreases. The mean crack spacing (L_r) on the contrary varies relatively less (slight decrease) as a function of temperature.

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