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The effect of reinforcement on early-age cracking due to autogenous shrinkage and thermal effects

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Abstract

The effect of reinforcement on early-age cracking in high strength concrete was investigated on laboratory scale. A temperature stress testing machine was used for simulating total restraint and for imposing different curing temperatures onto the concrete. The behaviour of the used high strength concrete was compared to a normal strength concrete. In order to separate thermal effects and autogenous shrinkage specimens were cured isothermally and semi-adiabatically. Further test variables were the reinforcement percentage (0%, 0.75%, 1.34% and 3.02%) and configuration (one reinforcement bar and four reinforcement bars). In order to visualise the crack formation in the early phase, cracks had been impregnated with fluorescent epoxy. The so obtained photos show that reinforcement can induce the formation of smaller cracks. These smaller cracks can postpone the moment at which major cracks are formed. Finally, a procedure is discussed for quantifying the effect of reinforcement decreasing the risk of through-cracking at early age. For that purpose a *strain enhancement factor* is introduced.

Keywords: Early-age cracking; Thermal effects; Autogenous shrinkage; Temperature stress testing machine; Reinforcement

1. Introduction

At early-age, concrete structures are subjected to deformations during the hydration process. If in the cooling phase contraction of the concrete is hindered, tensile stresses are generated in the structure. These tensile stresses cause cracking when exceeding the tensile strength of the hardening concrete. High strength concrete (HSC) is even more prone to early-age cracking, as it undergoes additional volume changes that are known as autogenous shrinkage.

For judging the risk of cracking in hardening concrete structures temperature criteria are still being used in practice. Although these criteria have proven to work in some cases, in many other cases they do not. These criteria do not, for example, considered the degree of restraint, which is in fact the factor that dominates the actual risk of cracking.

Particularly when dealing with low water/binder ratio concretes, in which the risk of cracking is significantly affected by the presence of autogenous shrinkage, temperature criteria do not apply. In essence, it is the strain capacity of the concrete that determines the risk of cracking. Moreover, it is well known that in case of deformation-controlled loading of the concrete the imposed strain prior to the moment of the formation of large cracks can benefit from the pre-peak deformational behaviour of the concrete.

So far, most studies on the risk of cracking in hardening concrete have been carried out on unreinforced concrete. In those studies any positive effect of the pre-peak tensile strain behaviour of the concrete has hardly been observed. Observations in practice have shown, however, that in case of heavily reinforced concrete the risk of cracking is much less than experiments with plain concrete suggest. Obviously, the presence of reinforcement enables the concrete to make its pre-peak strain capacity operational so that the moment of cracking is postponed.

Although this phenomenon cannot be considered as completely unexpected, it is not clear yet how important the role of reinforced is as far as the risk of cracking is concerned. Therefore, reinforced specimens have now been tested on laboratory scale. A temperature stress testing machine (TSTM) was used to simulate total restraint and different curing temperatures. In addition the

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development of concrete material properties, such as concrete compressive strength and E-modulus, have been monitored. In order to visualise the crack formation, individual cracks have been impregnated with fluorescent epoxy. After hardening of the epoxy the specimens were sliced up.

The experimental results should give answers to the following questions:

- How does reinforcement influence the moment of cracking and what is the effect of the reinforcement configuration and the reinforcement percentage in HSC?
- How can the effect of reinforcement on the risk of cracking be formulated and quantified in HSC?

2. Experimental programme

The stress development and the cracking behaviour of a fully restrained, reinforced concrete element was tested in a TSTM. The free deformations of plain concrete and reinforced concrete were measured in dummies. Table 1 shows the concrete composition of the tested HSC (w/c ratio = 0.33). In addition some experiments have been performed on NSC (w/c ratio = 0.5) in order to investigate the effect of additional autogenous deformations on cracking behaviour in reinforced specimens. Specimens of each experiment were cast from the same batch of concrete and cured semi-adiabatically. In order to investigate the effect of reinforcement on autogenous shrinkage HSC-specimens were also cured at 20 °C isothermally. All specimens were sealed.

The TSTM-specimens were tested with three different percentages of reinforcement, 0.75%, 1.34%, 3.02% and without reinforcement. Each reinforcement percentage was realised with two different configurations of rebars: 4 ∅ 6 or 1 ∅ 12, 4 ∅ 8 or 1 ∅ 16, and, 4 ∅ 12 or 1 ∅ 25, respectively (Table 2).

Table 1
Concrete composition per kg/m^3

	HSC [kg/m^3]	NSC [kg/m^3]
Water	125.4v	175.0
Cem III/ B 42.5 LH HS	237.0	–
CEM I 52.5 R	238.0	–
CEM I 32.5 R		350
Silicol SL (50/50 slurry)	50.0	
Lignosulphonate	0.9	
Naphtalene sulphonate	9.5	
Gravel 4–16 mm	973.5	
Gravel 4–8 mm		827.6
Sand 0–4 mm	796.5	1011.5
Slump flow [mm]	250	
Slump [mm]		30

Table 2
List of experiments

Number of tests	Concrete composition	Reinforcement configuration	Reinforcement percentage	Temperature development
2	HSC	–	–	Semi-adiabatic, 20 °C
1	HSC	1 ∅ 12	0.75	Semi-adiabatic, 20 °C
1	HSC	4 ∅ 6	0.75	Semi-adiabatic, 20 °C
1	HSC	1 ∅ 16	1.34	Semi-adiabatic, 20 °C
3	HSC	4 ∅ 8	1.34	Semi-adiabatic, 20 °C
1	HSC	1 ∅ 25	3.27	Semi-adiabatic, 20 °C
3	HSC	4 ∅ 12	3.02	Semi-adiabatic, 20 °C.
1	NSC	–	–	Semi-adiabatic
1	NSC	1 ∅ 16	0.89	Semi-adiabatic
1	NSC	4 ∅ 8	0.89	Semi-adiabatic
1	NSC	1 ∅ 25	2.18	Semi-adiabatic
1	NSC	4 ∅ 12	2.01	Semi-adiabatic

3. Test equipment

The TSTM is a horizontal steel frame in which hardening concrete specimens can be loaded in compression and in tension under various hardening conditions (Fig. 1) [1]. Both load-controlled and deformation-controlled experiments can be performed under pre-defined thermal conditions.

To perform experiments in tension, a dovetailed interlock is used between the concrete specimen and the frame. Two steel claws hold the dovetailed specimen. One of the claws is fixed to the frame, the other is placed on roller-bearings and can be moved with the hydraulic actuator. Loads are measured with a load cell.

The TSTM used for these experiments was used in combination with two ADTM-specimens. In the first dummy the free deformation of concrete was measured. This dummy was also used to create reference conditions for the temperature development during hardening for all other specimens. The temperature was measured with thermocouples inserted in the concrete at different positions immediately after casting. The temperature measurements were started within approximately 30 min after mixing.

Load-independent deformations were measured with LVDT's. They measured the length changes of the

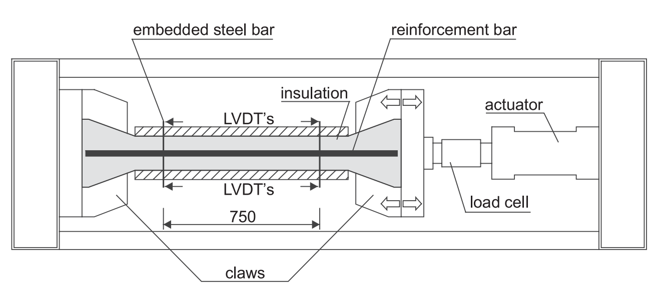

Fig. 1. The temperature stress testing machine (top view).

ADTM-specimens over a measuring length of 750 mm. This was done with the help of small steel bars perpendicular to the measuring direction. The bars were embedded in the specimen and passed the mould through holes (see Fig. 1). The second ADTM-specimen was reinforced in order to get an idea of the effect of reinforcement on the free deformation of reinforced specimens.

The cube compressive strength and the modulus of elasticity were tested at different early ages for monitoring the development of concrete properties. By using temperature controlled steel moulds, concrete cubes for the strength and prisms for the E-modulus were subjected to the same temperature history as the ADTM-specimens and the TSTM-specimen. The cube compressive strength was tested after 8 h, 24 h, 30 h, 48 h, 144 h and 28 days (672 h) (Table 3). The E-modulus was tested after 24 h, 72 h, 168 h and 28 days (672 h) (Table 4). All specimens were sealed.

Table 3
Cube compressive; strength mean value of the first and second test series

	Age [h]					
	8	24	30	48	144	672
HSC, cured at 20 °C	0.37	39.45	46.40	59.79	–	104.55
HSC, cured semi-adiabatically	1.52	66.28	83.42	90.62	97.12	100.68
NSC, cured semi-adiabatically	0.5	9.24	11.9	16.86	–	38.72

Table 4
Modulus of elasticity

	Age [h]			
	24	48	168	672
HSC, cured at 20 °C	27.8	32.6	37	39.8
HSC, cured semi-adiabatically	36.4	39.2	39.5	39.9
NSC, cured semi-adiabatically	25.6	29.3	32.5	32.9

4. Experimental results

4.1. Free deformation

During the hydration process cementitious materials undergo volume changes of different nature. Laboratory scale specimens are sealed to avoid moisture loss to the environment. In a first approximation load-independent volume changes (ε_{tot}) can be written as the sum of autogenous phenomena (ε_{aut}) and thermal deformations (ε_T):

$$\varepsilon_{tot} = \varepsilon_{aut} + \varepsilon_T \tag{1}$$

Deformation measurements were started at the moment that the concrete was stiff enough to keep the embedded steel bars in position to which the LVDT's were fixed outside the specimen. With time and experience, this moment could be fixed at an earlier time so that more information about the deformations in the very early phase could be obtained. The deformations were zeroed at the moment when stresses were built up in the TSTM.

4.1.1. Autogenous deformations

Under isothermal curing conditions only autogenous deformations have been registered in HSC. Under the assumption that the curing temperature is absolutely constant the rebars should not be subjected to significant deformations. Depending on the reinforcement percentage and configuration, restraint reached values up to 30% of the total free deformation at 20 °C (Fig. 2). Fig. 2 gives the concrete strain measured (left) and calculated (right) in plain specimens and differently reinforced specimens. The calculation is based on the transfer of forces via bond [2]. In order to account for the development of bond between concrete and rebar use was made of a bond stress–slip relation found in

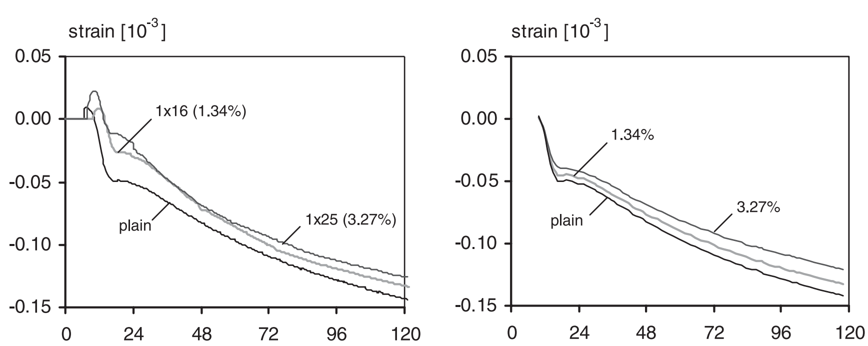

Fig. 2. Concrete strain measured (left) and calculated (right) in a plain specimens and reinforced specimens cured at 20 °C isothermally.

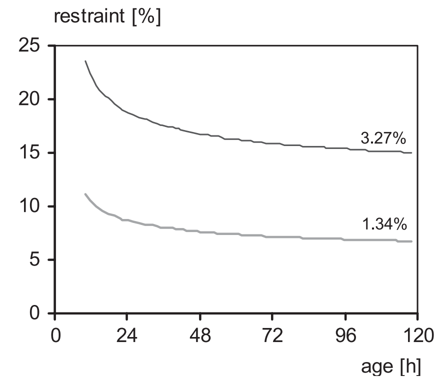

Fig. 3. Restraint of concrete strains caused by two different reinforcement ratios.

pull-out tests at very early ages [3]. The results show that the calculation describes the measurements very well. The concrete deformation decreases with increasing reinforcement percentage. The reinforcement configuration has no influence. Due to the development of bond strength it can be seen that the restraint of autogenous deformations is the most significant in the very early stage (Fig. 3).

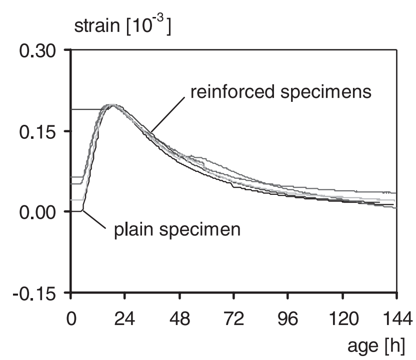

Fig. 4. Free deformation in reinforced and plain NSC-specimens cured semi-adiabatically.

4.1.2. Thermal deformation

Under semi-adiabatic curing conditions, concrete deformations are ruled by the hydration-induced thermal strains. As the thermal dilation coefficient of rebars is about the same as for concrete (except for the early phase) the influence of reinforcement on the thermal deformation is relatively small.

In NSC it was found that the measured deformations of reinforced specimens were about the same as for plain specimens. The differences of the experimental results are only due to the moment of starting the experiment. Assuming that the swelling is the same in all experiments the curves plotted in Fig. 4 are almost identical. This result can be explained by the fact that autogenous shrinkage plays a minor role in a concrete with a w/c-ratio of 0.5.

Fig. 5 shows the mean value of free deformations measured in all experiments with scatter on plain dummies (left) and reinforced ADTM-specimens (right) in HSC cured semi-adiabatically. The total free deformation measured after 144 h in reinforced specimens ($\varepsilon_{c,\text{reinforced}}$ (144) $= 0.455 \times 10^{-6}$), is insignificantly smaller than in plain specimens ($\varepsilon_{c,\text{plain}}$ (144) $= 0.468 \times 10^{-6}$). On average reinforcement restrains the free deformation of the tested HSC only by 2.8% of the total deformation. It thus appears that the presence of reinforcement does not influence the free deformation of the samples very much.

4.2. Stress development

Already at early age, stresses can be generated if load-independent deformations are restrained. If the generated stresses exceed the developed tensile strength of the concrete, it cracks. In our case differently reinforced specimens were tested and they were compared to a unreinforced specimen. Reinforcement, however, is only activated if the concrete moves relatively to the steel bars, i.e. in the case that the concrete cracks. Under isothermal curing conditions (20 °C) the development of stresses in the tested HSC did not exceed the tensile strength. Consequently, the concrete stress was about

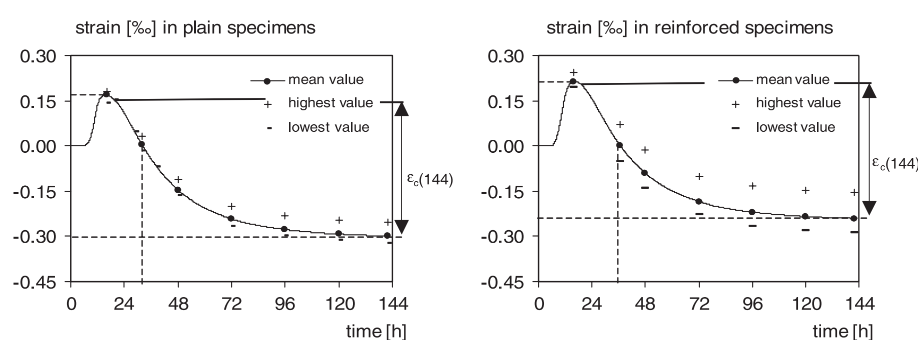

Fig. 5. Mean value of free deformation measured in all experiments with scatter on plain ADTM-specimens (left) and reinforced dummies (right), HSC cured semi-adiabatically.

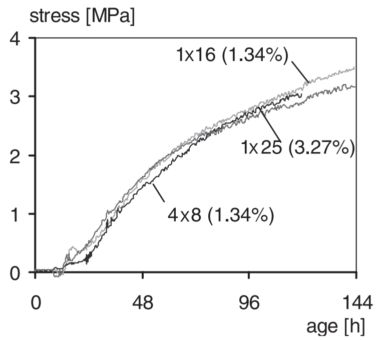

Fig. 6. Concrete stresses in HSC specimens with reinforcement ratios cured at 20 °C.

the same in all specimens whether they were reinforced or not (Fig. 6).

Specimens cured semi-adiabatically cracked (for the first time) in a period between 27 and 48 h after casting. In HSC it was found that specimens reinforced with four rebars cracked significantly later than plain specimens (Fig. 7). There was no significant difference found for the two investigated reinforcement percentage (4 ∅ 8 versus 4 ∅ 12). HSC-specimens reinforced with one rebar cracked at about the same age or only little later than plain specimens.

Under sealed conditions it can be assumed that there are almost only thermal deformations causing stresses in NSC. For this reason it is quite simple to estimate concrete stresses in reinforced NSC-specimens (Eq. (2)). In Fig. 8 the stress development measured in a plain specimen is compared to the stresses measured in a specimen reinforced with one (Fig. 8, left) and with four rebars (Fig. 8, right). In addition the concrete stresses are shown that have been calculated with Eq. (2) for both reinforced specimens.

$$\sigma_c = \frac{A_{tot} \cdot \sigma_{tot} - A_s \cdot \sigma_s}{A_c} \tag{2}$$

Fig. 8 shows that the stress development of reinforced and the plain specimen is about the same. This was expected under the assumption that the thermal dilatation coefficient of steel and concrete are about the same (in the range of $10–12 \times 10^{-6}$). As the concrete stress in a reinforced specimen reaches the concrete tensile stress earlier than in a unreinforced specimen the specimen reinforced with one rebar cracks earlier than the plain specimen.

In a NSC-specimen reinforced with four rebars the same observation has been made as in HSC. Crack formation indicated by an unsteady stress curve postpones the first bigger crack some hours in comparison to

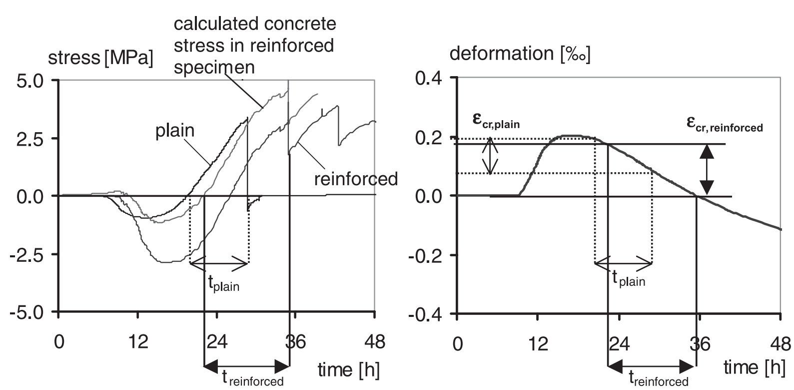

Fig. 7. Stress development in a reinforced (4 ∅ 12) and a plain HSC-specimen cured semi-adiabatically. Tensile strain capacity $\varepsilon_{cr,reinforced} > \varepsilon_{cr,plain}$.

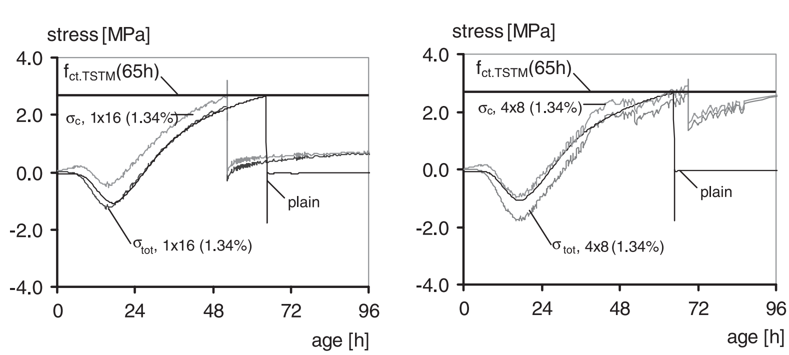

Fig. 8. Stress development in NSC-specimens reinforced with one (left) and four rebars (right) compared to a plain specimen under semi-adiabatic curing condition.

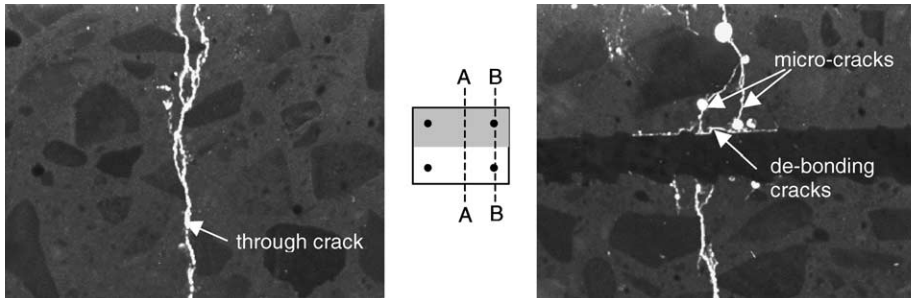

Fig. 9. Crack pattern in reinforced specimens in the middle of the specimen (Section A-A, left) and in the vicinity of the rebars (Section B-B, right).

the plain specimen. The stress drop in the specimen reinforced with four rebars measures less than half of the stress drop in a plain specimen. In the specimen reinforced with one rebar the stress drop is about the same magnitude as the one in the plain specimen.

4.3. Crack formation

In order to get more information about the fracture process inside a HSC-specimen reinforced with $4\varnothing 12$, two cracks have been impregnated with fluorescent epoxy shortly after cracking. The specimens were sliced up when the epoxy was hardened. Fig. 9, left, illustrates the formation of splitting cracks in the vicinity of the rebars. These cracks are smaller compared to the cracks in the middle of the specimen (Fig. 9, right). It is assumed that the formation of the smaller cracks enhance the strain capacity of the TSTM-specimen before the occurrence of the first through crack.

It must be noted that a simple method was used to visualise internal cracks that has its limitations. When no cracks can be seen it does not necessarily mean that they do not exist. The only conclusion which can be drawn is that within the limitations of the present impregnation method no other cracks were found than those visible on the picture.

5. Strain enhancement factor

In literature [4,5] different criteria are used in order to quantify the risk of cracking in young concrete. In plain concrete a stress criterion is mostly applied, relating the stress at the moment of cracking to the tensile strength at the same moment (e.g. [6]).

Our experimental results revealed that the presence of four rebars postponed the moment of cracking. Comparing the results with specimens reinforced with $4\varnothing 8$ versus $4\varnothing 12$, the reinforcement ratio did not seem to have a significant influence on the postponement of the moment of cracking. In both cases the stress curve bends before the first through-crack occurs. This bending of the stress curve is interpreted as being caused by micro-

Table 5
Statistical parameters for calculating the strain enhancement factor

Test	ε_{cr} $[10^{-6}]$	Mean value $[10^{-6}]$	Standard deviation $[10^{-6}]$
Plain	91 92	92	7
$4\varnothing 8$	161 160 188	170	16
$4\varnothing 12$	168 198 142	169	28

cracking. In order to judge the positive effect of micro-cracking, the tensile stress-inducing strains until the moment of through-cracking in a reinforced specimen $\varepsilon_{cr,reinforced}$) and a plain specimen ($\varepsilon_{cr,plain}$) are compared (Table 5). On average this strain in a reinforced specimens is 108×10^{-6} larger than in a plain specimens. This extra strain equals 1.85 times the strains in plain specimens. Schematically this is shown in Fig. 7.

Table 5 summarises the mean values and the standard deviations of the strains from the moment of zero-stress in concrete until the moment of through-cracking, i.e. the tensile strain capacity prior to cracking. This has been done for the two plain specimens and for the two series of reinforced specimen, each series consisting of three specimens. As mentioned in the foregoing, there is hardly any difference in the increase of the tensile strain capacity of the two reinforced series compared to the plain concrete specimens. The mean value of the tensile strain capacity of all the reinforced specimens is, therefore, about equal to that of the two individual reinforced series, viz. $\varepsilon_{cr,reinforced,mean} = 170 \times 10^{-6}$ with a standard deviation of $\sigma = 20 \times 10^{-6}$. The corresponding values of the plain specimens are $\varepsilon_{cr,plain,mean} = 92 \times 10^{-6}$ and $\sigma = 7 \times 10^{-6}$ (based on only two samples). The extra tensile strain capacity of the reinforced specimens can be expressed with a strain enhancement factor η. This strain enhancement factor refers to the mean values of the tensile strain capacity of plain and reinforced concrete. It holds:

$$\eta_{\text{mean}} = \frac{\varepsilon_{\text{cr,reinforced,mean}}}{\varepsilon_{\text{cr,plain,mean}}} \tag{3}$$

with the strain enhancement factor the positive effect of micro-cracking on the moment of the occurrence of large cracks in reinforced concrete, i.e. the postponement of this moment, can be quantified.

6. Discussion and conclusion

Based on the experimental results obtained in this study, the following answers can be given to the questions put forward in Section 1.

How does reinforcement influence the moment of cracking and what is the influence of the reinforcement configuration and the reinforcement percentage?

HSC specimens reinforced with one rebar cracked at about the same moment and in about the same way as plain specimens. Specimens reinforced with four rebars cracked later than plain specimens. By impregnating cracks and sawing the specimens it was found that in the vicinity of the rebars, cracks split into smaller cracks. These micro-cracks could lead to an increase of the strain of a reinforced concrete specimen before creating a through-crack.

How can the influence of reinforcement on the risk of cracking be formulated?

In the introduction reference was made to temperature criteria that are used in the engineering practice for judging the risk of cracking in hardening concrete. Temperature differences between new and old concrete, or between the surface layer and the core of a concrete element, should not exceed 15–25 K. Assuming a coefficient of thermal expansion (α_T) of 10^{-5}/K these criteria correspond with an imposed strain of 150×10^{-6}–250×10^{-6}. In the experiments with unreinforced concrete the imposed tensile strain at failure was 92×10^{-6}. In case of reinforced concrete the mean values of the imposed tensile strain was found to be 170×10^{-6} and 169×10^{-6} for specimens reinforced with $4 \varnothing 8$ and $4 \varnothing 12$, respectively (see Table 5) after the moment of zero stress in concrete. Obviously the presence of four reinforcing bars increased the tensile strain capacity by about 108×10^{-6} compared with plain specimen (see Table 5). In terms of temperature strains this would mean that an additional strain could be accommodated equal to 10.8 K (with $\alpha_T = 10^{-5}$ /K).

The increase in tensile strain capacity can be attributed to the formation of smaller cracks, which could be visualised by impregnating these cracks with fluorescent epoxy. For quantification of the extra tensile strain capacity a strain enhancement factor η has been proposed. This factor indicates the increase of the tensile strain capacity until cracking of reinforced concrete compared with that of plain concrete. When referring to the mean values of the tensile strain capacity of reinforced and plain concrete, a strain enhancement factor was found of $\eta_{\text{mean}} = 1.8$ for the HSC.

Acknowledgements

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