MINIMUM REINFORCEMENT IN RC BEAMS

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SUMMARY

It is commonly agreed that before failure the RC structure must give a warning by cracking and visible deflections. This constitutes the requirements for the minimum reinforcement ratio. Research results suggest that minimum reinforcement ratio is member size dependant. The exact tendency is, however, not clear. In the lack of experimental data, extrapolation of member size dependence beyond the tested size range often takes place. Considering this situation, extensive test series on beams with minimum reinforcement ratios are initiated, varying the member depth in a wide range. In this paper preliminary test results are discussed. They show that the minimum reinforcement ratio of small beams may be reduced, with respects to standard provisions. Further tests on beams with lower reinforcement ratios and more brittle concrete should permit a formulation of member size dependence law for minimum reinforcement ratio.

Keywords: Minimum Reinforcement, Finite Element Analyses, Size Effect, Fracture Mechanics, Reinforced Beams

1. INTRODUCTION

In EC2 (1992), CEB-FIP Model Code 1990 (1993) and ACI 318 (1989) the minimum reinforcement ratio is member size independent. Only few codes as e.g. the Norwegian Code 3473 (1992) and some recent numerical studies (Carpinteri, 1992; Ozbolt, 1995) suggest that the minimum reinforcement ratio is size dependent and that in some cases the codes might be unsafe (Fig. 1). Different models contradict in their predictions of member size dependence of minimum reinforcement ratio, however, they all agree on the reasons for this phenomenon.

Size dependence of minimum reinforcement ratio is likely to be a result of the decreasing energy dissipation capacity of concrete (increasing brittleness) for increasing member depth. This is the result of strain localisation in the fracture zone of limited, member size independent length. Research done on plain concrete proofs that with increasing concrete strength its brittleness increases. Also this effect is size depended. The question remains open whether in RC similar shift to more brittle failure mode should be expected with increasing concrete strength. In order to provide the answer to this question and to verify the provisions of design standards with respect to the size dependence of the minimum reinforcement ratio both experimental and numerical studies were carried out.

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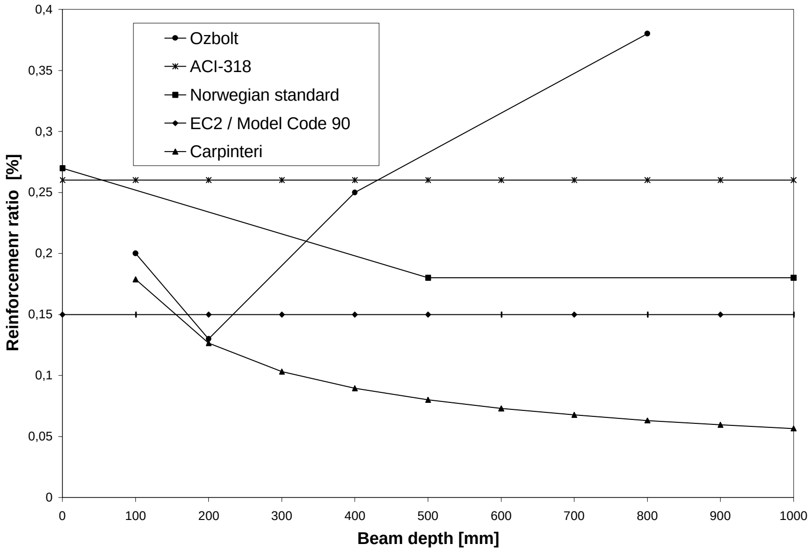

Fig. 1. Influence of the beam depth to the minimum reinforcement

2. EXPERIMENTS

To get information about the structural behaviour of low reinforced concrete beams of different sizes, tests were carried out at the Institute for Construction Materials (University of Stuttgart). Depth of the members is varied from h = 0.125 m to 1.50 m. In the following results are reported for h = 0.125 m to 0.50 m.

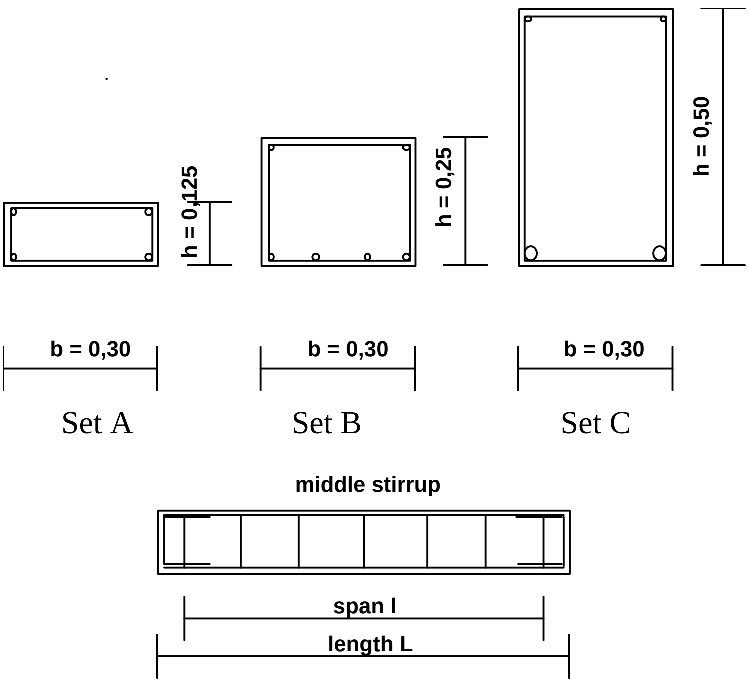

Fig 2. Geometry and reinforcement of beams

The reinforcement ratio was chosen according to EC2 (1992) requirements for minimum reinforcement ratio: A_s / A_c = 0.15 %. The reinforcement was designed as close as possible to practise and as often, stirrups and constructive reinforcement in the compression zone of the concrete were used. In order to initiate the major crack in the middle of the beam one stirrup was arranged exactly in the vertical symmetry axis of the beam. The spacing of the stirrups was taken according to the minimum shear reinforcement requirements of EC2 (1992) in order to avoid shear failure. The concrete cover was not scaled and was fixed to c = 30 mm. The ratio between span and beam depth was chosen equal to l / h = 6, while the ratio between total length and depth equalled L / h = 7. Geometry of beams is shown in Fig. 2 and specified in Tab. 1.

Table 1. Specimens geometry

Set	h [m]	b [m]	l [m]	L [m]	r_l [%]	reinforcement	stirrups	spacing s [m]
A	0.125	0.30	0.75	0.875	0.15	2 ϕ 6	ϕ 6	0.125
B	0.250	0.30	1.50	1.750	0.15	4 ϕ 6	ϕ 6	0.150
C	0.500	0.30	3.00	3.500	0.15	2 ϕ 12	ϕ 6	0.200

2.1 Material properties

2.1.1 Concrete

In the preliminary test series normal strength concrete C25/30 was used. The maximum aggregate size was 16 mm. Specimens were kept in the mould for 20 days and extensively cured in order to avoid cracks due to shrinkage of concrete. Tab. 2 gives the mean values of the concrete characteristics for sets A, B and C: compressive strength f_c, splitting tensile strength $f_{ct,sp}$, bending tensile strength $f_{ct,fl}$ and uniaxial tensile strength (MC90,1993) f_{ctm}. To obtain the value of the fracture energy G_f tests according to RILEM (1985) were carried out additionally.

Table 2. Concrete characteristics

Set	f_c [N/mm²]	$f_{ct,sp}$ [N/mm²]	$f_{ct,fl}$ [N/mm²]	f_{ctm} [N/mm²]	G_f [N/m]
A	31.80	3.03	3.07	2.73	
B	31.80	3.03	3.07	2.73	
C	33.60	3.13	3.18	2.73	89.7

2.1.2 Steel

The members were reinforced with hot-rolled ribbed bars. The mean values of yield strength f_y, ultimate strength f_t, ultimate strain e_{su} and Young's modulus E_s are shown in Tab. 3 for both bar diameters used.

Table 3. Steel properties

f	f_y	f_t	e_{su}	E_s
mm	[N/mm²]	[N/mm²]	[%]	[N/mm²]
6	578.3	638.6	16.3	200366
12	580.0	631.5	23.0	198250

2.2 Test - Setup

The beams were tested in three-point-bending. The load was applied with a load jack at mid-span. Steel roller bearings assured free horizontal translation and rotation at both supports. Test were carried out in deformation control using the mid-span deflection as controlling parameter. The displacement rate was kept constant and in the first state of loading it was equal to v = 0,002 mm/s. After formation of the last bending crack the loading rate was increased to v = 0,01 mm/s. The force was measured with an internal load cell. The displacements of the hydraulic cylinder was recorded as well. The crack opening and the mid-span deflection were measured with electric resistance transducers (LVDT). The crack width was measured at three levels over the member depth. The gauge length was $\ell_b = 100$ mm. The location of the LVDTs used for crack widths registration is shown in Fig. 3. All measurements were monitored and recorded with a frequency of 2 Hz.

2.3 Results

2.3.1 General

For each combination of variables (set A,B,C) two tests were carried out. The load-crack opening curves measured at the level of the reinforcement are shown in Fig. 4, 5, and 6. This crack opening is assumed to be equal to the displacement measured with LVDT 1.

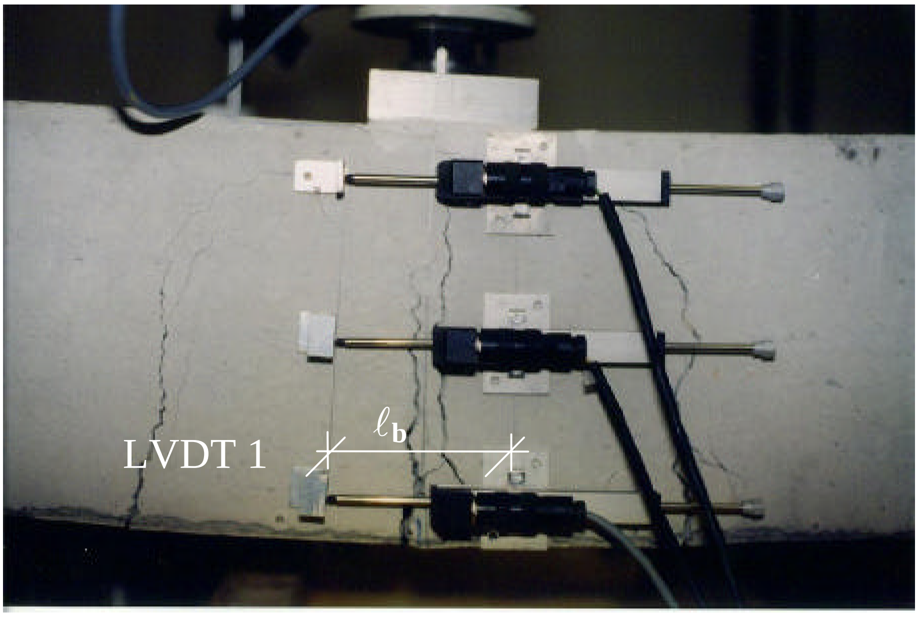

Fig. 3. Measuring LVDT

The yield load and the ultimate load calculated assuming the material models according to EC2 (a bilinear stress-strain curve of steel and a parabolic-rectangle stress-strain curve of the concrete, maximum concrete compressive strain ε_{cu} = -0.0033) and using the actual material strength are shown in Fig. 4, 5, and 6 as well.

2.3.2 Set A (h = 0,125 m)

In both tests the first crack occurred at a load level of ~10 kN, which corresponds to a concrete tensile strength of 2.5 MPa, when calculating the cracking load according to the theory of elasticity. The measured ultimate load was ~27 kN. After the peak load was reached the load suddenly dropped to ~14 kN. Subsequently it slightly increased and reached ~18 kN. It remained at this level until rupture of the reinforcement. In total 3 cracks formed and their location corresponded with the stirrup positions.

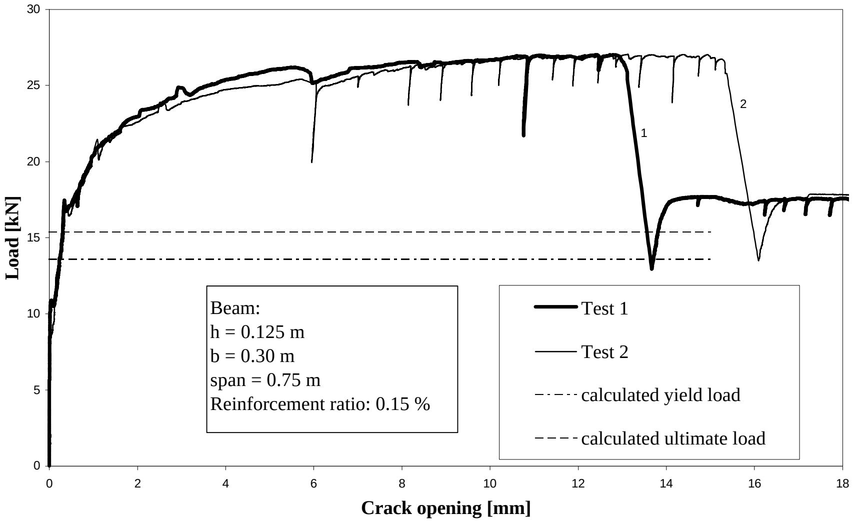

Fig. 4. Force-crack opening curve of beam set A (h = 0,125 m)

2.3.3 Set B (h = 0,25 m)

When testing beam set B in both tests the first crack occurred at the level of ~19 kN, which corresponds to a concrete tensile strength of 2,5 MPa. The maximum load reached ~48 kN. After reaching the peak load the load suddenly dropped to ~31 kN. Similar as in the test set A, the load recovered and reached ~34 kN before the steel failed. In Test 2 the steel failed at a load level of ~42 kN, after a smooth softening branch was observed.

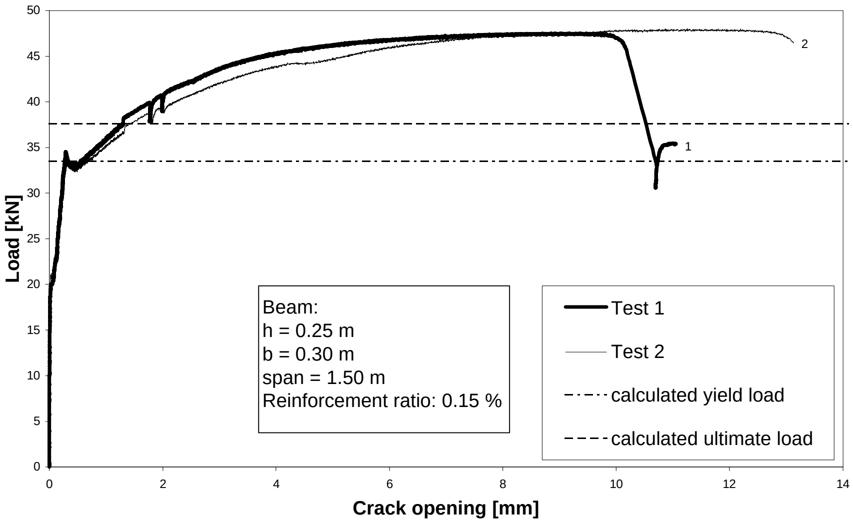

Fig. 5. Force-crack opening curve of beam set B (h = 0,25 m)

In total 3 cracks occured. The location of the cracks correspond to the location of the stirrups as it was seen in test set A.

2.3.4 Set C (h = 0,50 m)

Beam set C cracked at a load level of ~55 kN (Test1) and ~50 kN (Test 2). It corresponds to a calculated tensile strength of 3,6 MPa (Test1) and 3,3 MPa (Test2) respectively. The maximum load was equal to ~89,0 kN. In both cases after a relatively long softening branch the steel ruptured. In total 8 cracks were formed. As in the other test sets the crack location corresponded with the stirrup positions.

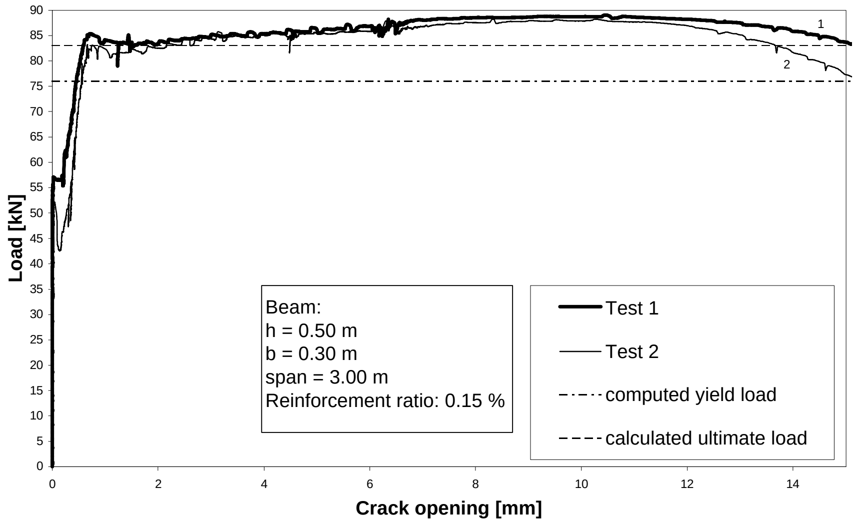

Fig. 6. Force-crack opening curve of beam set C (h = 0,50 m)

3. DISCUSSION AND CONCLUSIONS

All tested beams showed a very ductile behaviour. After the first crack occurred the load slightly decreased, but was easily to a level higher then the cracking load. This satisfies the classical conditions of minimum ductility. The failure could be noticed far in advance by large deflections and very large crack openings (total deflections were 30 mm for beam set A, 35 mm for beam set B and 45 mm for beam set C).
In order to compare the structural ductility of the beams with different sizes the deflection at failure is divided by the span. The ratio is equal to 0,40 for beam set A, 0,23 for beam set B and 0,15 for beam set C. This proves, that with increasing size the structural ductility decreases, hence the member brittleness increases.
Size dependence of the concrete contribution to the load bearing capacity can be evaluated comparing the measured ultimate load with the value calculated neglecting size effect. The so-called hyper-strength (Cossu and Pozzo, 1990) equals 1,70 for beam set A, 1,30 for beam set B and 1,05 for beam set C. Hence, the load bearing capacity is increasing with decreasing member size. This is in full agreement with the asumptions of the theory of Linear Elastic Fracture Mechanics (Hillerborg, 1989).
Both, beam set A and beam set B show a steep descending branch. The load recovers and stabilise at a new level. After this steel finally failed in tension. The reason for the drop of the loads is most probably subsequent failure of single reinforcing bars.

Since beams loaded in bending show a increasing ductility and a rising concrete contribution with decreasing member size, the minimum reinforcement ratio could be reduced with decreasing beam size. The tests of Bosco and Carpinteri (1992) and the numerical results of Ozbolt (1995) shown in Fig. 1 suggest a contradictory behaviour. This could be the result of other reinforcement arrangement, i.e. use of stirrups and compressive reinforcement in the own tests.

In order to verify the suggested proportionality between member size and minimum reinforcement ratio, tests will be carried out with lower amount of reinforcement. Furthermore, tests on larger members (h = 0.50 m to 1.50 m) are currently under preparation. The effect of increased concrete strength on the member size dependence of minimum reinforcement ratio also need to be investigated in the future.

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