BOND BEHAVIOUR OF RIBBED BARS AT INELASTIC STEEL STRAINS

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SUMMARY

Bond between concrete and reinforcement in RC members has to guarantee small crack width in SLS, enough rotation capacity of plastic hinges in ULS and sufficient bearing capacity of anchorages and lap splices for short anchorage and splice lengths. Until now the knowledge of bond behaviour for plastic steel strains is still incomplete. Therefore tests on RC columns loaded in uniaxial tension were performed. The results indicate that the tension stiffening beyond yielding is significantly influenced by steel ductility, especially for low reinforcement ratios. A rational bond model that accounts for yielding of steel and the bar surface geometry is developed and will be implemented in a non-linear FE-program. Simulations of RC members should allow to optimise rib pattern of reinforcement in the view of the fundamental requirements on bond.

Keywords: Bond, Anchorage, Reinforcement, Tension stiffening, Ductility

1. INTRODUCTION

The behaviour of reinforced concrete (RC) structures is significantly influenced by bond. The bond should fulfil the following requirements: **a)** In the serviceabilty limit state (SLS) the width of cracks and the overall deformations of RC members should be smaller than allowable values. This requires small crack spacing and a high contribution of concrete between cracks i.e. a low ratio between the mean steel strain e_{sm} (mean embedded bar strain) and the steel strain e_{sr} at the crack (bare bar strain). This can be obtained by a high bond stiffness (small slip values corresponding with high bond stresses) and a sufficiently high bond strength. **b)** In the ultimate limit state (ULS), i.e. at inelastic steel strains, the ratio e_{sm} / e_{sr} should be large to ensure ductile behaviour of RC members with large rotation capacity at plastic hinges. To that end the bond resistance after passing the yield strain should be low. **c)** In the area of anchorages and lap splices of reinforcement high bond strength should come together with small splitting forces to guarantee short anchorage and lap lengths. These requirements partly contradict each other and cannot be equally satisfied at the same time. A compromise between the different requirements is needed, hence bond should be optimised. The bond behaviour of deformed bars depends significantly on the projected rib area f_R which is defined as the ratio between the rib bearing area and the rib shearing area (Rehm, 1961). According to prEN 10080 (1998) the projected rib area of bars with

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diameter $d_s \geq 11$ mm must be $f_R \geq 0.056$. This requirement, as well as the requirements for the rib patterns of deformed bars used currently in Europe and in the US, mainly take into account conditions **a)** and **c)**. They ensure good bond behaviour at SLS and high capacity of anchorages and lap splices, however at inelastic steel strains the contribution of concrete between cracks is likely to be too high, thus reducing the rotation capacity of plastic hinges.

In the course of this investigation the own test results and references from the literature (e.g. Tepfers, Olsson, 1992) are used to verify a rational bond model (Den Uijl, Bigaj, 1996), which takes into account yielding of steel and, with proposed extension (Mayer, 1998), also the bar surface geometry (f_R). After verification the bond model will be implemented in a non-linear FE-program NELIN 2 (Ozbolt, Mayer, 1997; Mayer, 1998). Using this program parametric studies of RC member behaviour in SLS and ULS will be performed that should allow to optimise bond (surface geometry - f_R) of ribbed reinforcement in the view of the above mentioned fundamental requirements.

2. EXPERIMENTAL INVESTIGATION

2.1 Investigated parameters

In Tab. 1 the test series are shown.

Bar diameter d_s [mm]	Reinforcement percentage r [%]	Steel class acc. MC 90	Concrete grade	Specimen geometry $a \times b \times l$ [m]
6	0.57	B	B 25	0.20 x 0.20 x 2.30
	0.85	A		
	1.13	A		
12	0.28	A	B25	0.40 x 0.40 x 2.50
	0.50	A		0.30 x 0.30 x 2.50
	0.50	B		
	0.75	B		
	1.00	A		
	1.00	A	B 45	
	1.50	A	B 25	
16	0.50	A	B 25	0.40 x 0.40 x 2.70
	0.50	S	B 45	
	0.75	A	B 25	
	1.00	A		
	1.00	A	B 45	
25	1.09	A	B 25	0.30 x 0.30 x 2.90
	1.23	A		0.40 x 0.40 x 2.90

Tab. 1: Varied parameters

The stiffness of tension members loaded up to rupture of the reinforcement is investigated by 34 tests. The tests were carried out varying reinforcement percentage ρ, bar diameter d_s, steel ductility (ductility classes B, A and S according to CEB-FIP Model Code 90 (MC 90), and concrete strength (see Tab. 1). For each combination of parameters two tests were performed.

2.2 Material properties

The average characteristics of concrete measured after 28 days on standard specimens cast and stored together with the test specimens were: for B 25 - $f_{cm,cube}$ = 30.4 MPa, f_{ctm} = 2.42 MPa, E_{cm} = 30691 MPa; for B 45 - $f_{cm,cube}$ = 64.7 MPa, f_{ctm} = 3.53 MPa, E_{cm} = 37407 MPa. The average characteristics of the bars d_s = 12 and 16 mm measured in standard tests are given in Tab. 2. The strains were exactly recorded up to $e_s \approx 6.0$ %. The values of A_{gt} (uniform elongation at peak stress) were measured after bar rupture.

Bar diameter d_s [mm]	Steel class acc. MC 90	f_y [MPa]	f_t [MPa]	f_t/f_y [-]	E_s [Mpa]	e_{sh} [%]	A_{gt} [%]
12	B	567	597	1.05	209000	-	~3.3
	A	532	587	1.10	196000	2.4	~6.40
16	A	519	588	1.13	203000	1.50	~8.50
	S	539	625	1.16	202000	2.10	~10.75

Tab. 2: Poperties of reinforcing steel (mean values)

2.3 Specimen geometry and test setup

The steel bars were cast in the square cross section of the specimen with a related concrete cover $c / d_s \approx 2.4$. Stirrups (d_s = 8 mm) were spaced at a distance equal to the crack spacing calculated according to MC 90. To avoid rupture of the reinforcement in the unbonded (free) length, the load introduction zones at the end of the specimens were additionally reinforced with hooked bars and the load was partly transmitted to the tested bars by splices.

The test setup is given in Fig. 1. Tensile force was applied with a hydraulic jack with a total capacity of 5000 kN. Tests were carried out in displacement control (mean loading rate v_{B1} = 0,01 mm/s up to yielding of the steel, v_{B2} = 0,02 mm/s up to rupture of the steel). The force was measured by an external load cell with a total capacity of 1000 kN. The overall elongation of the specimen was recorded with four displacement meters (LVDT with a gauge length = 2000 mm). Over the length equal to the calculated crack spacing the elongation was registered using four additional (Crack-) LVDTs. Furthermore the crack width was measured with four other (Crack-) LVDTs with a gauge length of 100 mm. To determine the location of the eight (Crack-) LVDTs the specimens were loaded until three cracks occured. After unloading, the LVDTs were applicated and specimens were further loaded until failure. The measurements were automatically recorded during the test using a digital amplifier (Spider 8) and were processed with a data acquisition system (CATMAN-PC).

In the following the results for ribbed bars with a diameter of d_s = 16 mm (reinforcement percentage r = 0,5%; 4 bars) are discussed in more detail. These tests are particularly interesting, because in this case at one bar of each specimen 30 post-yield strain gauges (TML) were placed in order to measure steel strain redistribution in vicinity of a (pre-formed) crack. Yet, a detailed information about the local bond behaviour in the post-yield range of ribbed reinforcement was obtained. The strain gauges were placed in grooves, pre-formed in the area of the longitudinal ribs and isolated against water and mechanical destruction. At each location two strain gauges were applicated on opposite sides of the bar. Due to the grooves the projected rib area was reduced from $f_R \approx 0.0725$ to $f_R \approx 0.0695$ and the cross-section of the bars from $A_s \approx 197.1$ mm^2 to $A_{s,g} \approx 186$ mm^2, respectively.

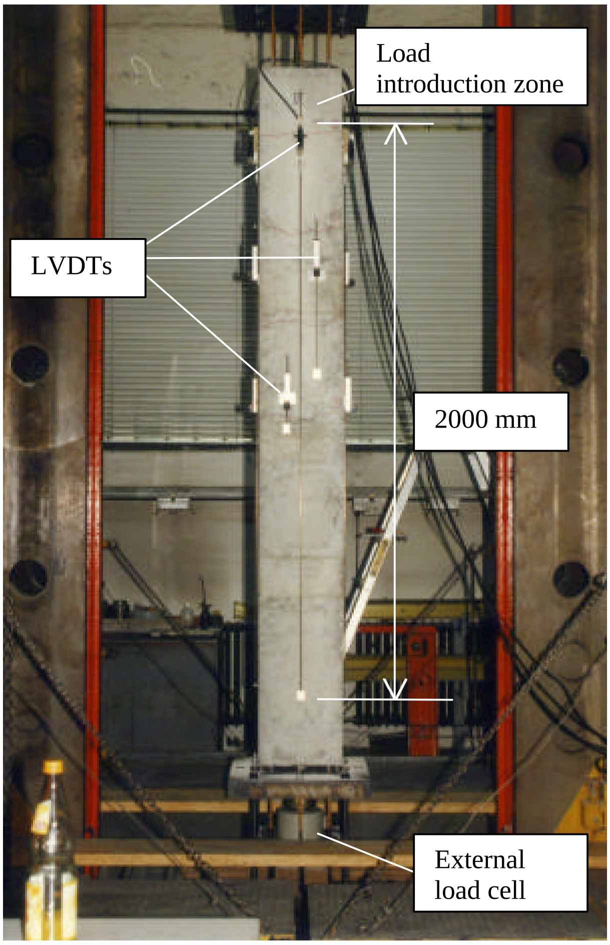


Fig. 1: Test setup

2.4 Test results

2.4.1 Stress-(mean) strain behaviour in the elastic steel strain range

The s_s-e_{sm}-behaviour in the elastic steel strain range (up to strains of 0.24%) of the specimens reinforced with bars $d_s = 16$ mm (reinforcement percentage $r = 0.5\%$) for different concrete grades and the bare bar response are compared in Fig. 2. The stress and strain values are determined from the measurements of the load and of the overall elongation (gauge length = 2000 mm).

With increasing concrete strength the stiffness of the tension member increases. Results of the performed tests confirm the well-known dependence of tension stiffening on reinforcement percentage, bar diameter and concrete strength in the elastic steel strain range.

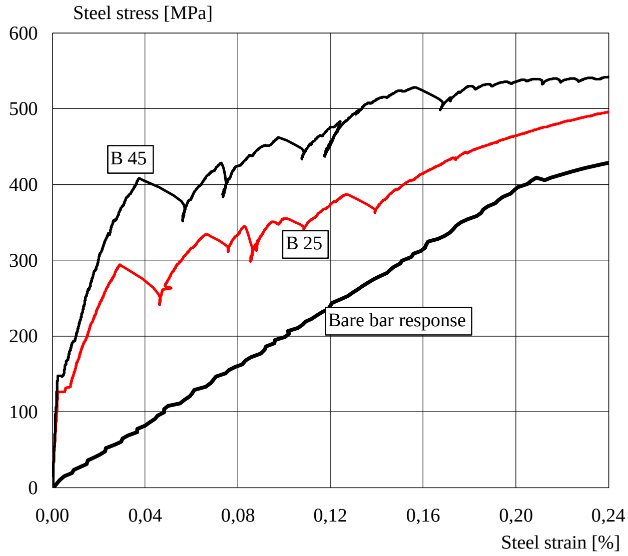


Fig. 2: Influence of concrete grade on the s_s-e_{sm}-behaviour for a reinforcement percentage r = 0.5%; comparison with bare bar response

2.4.2 Behaviour in the post-yield range of steel

The post-yield behaviour is described employing the locally measured strains (post-yield strain gauges) and using the ratio between the mean steel strain e_{sm} to the steel strain at the crack e_{sr} (e_{sm} / e_{sr}) and the steel strain at the crack e_{sr} to evaluate the contribution of concrete between cracks. In Fig. 3 the measured steel strains are plotted as a function of the distance from the pre-formed crack for different load stages.

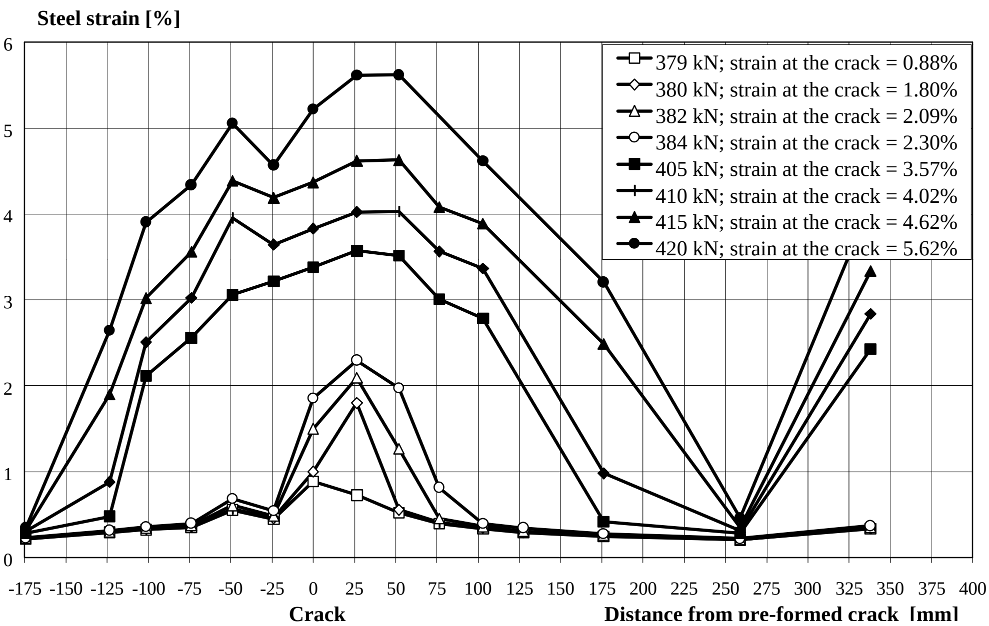
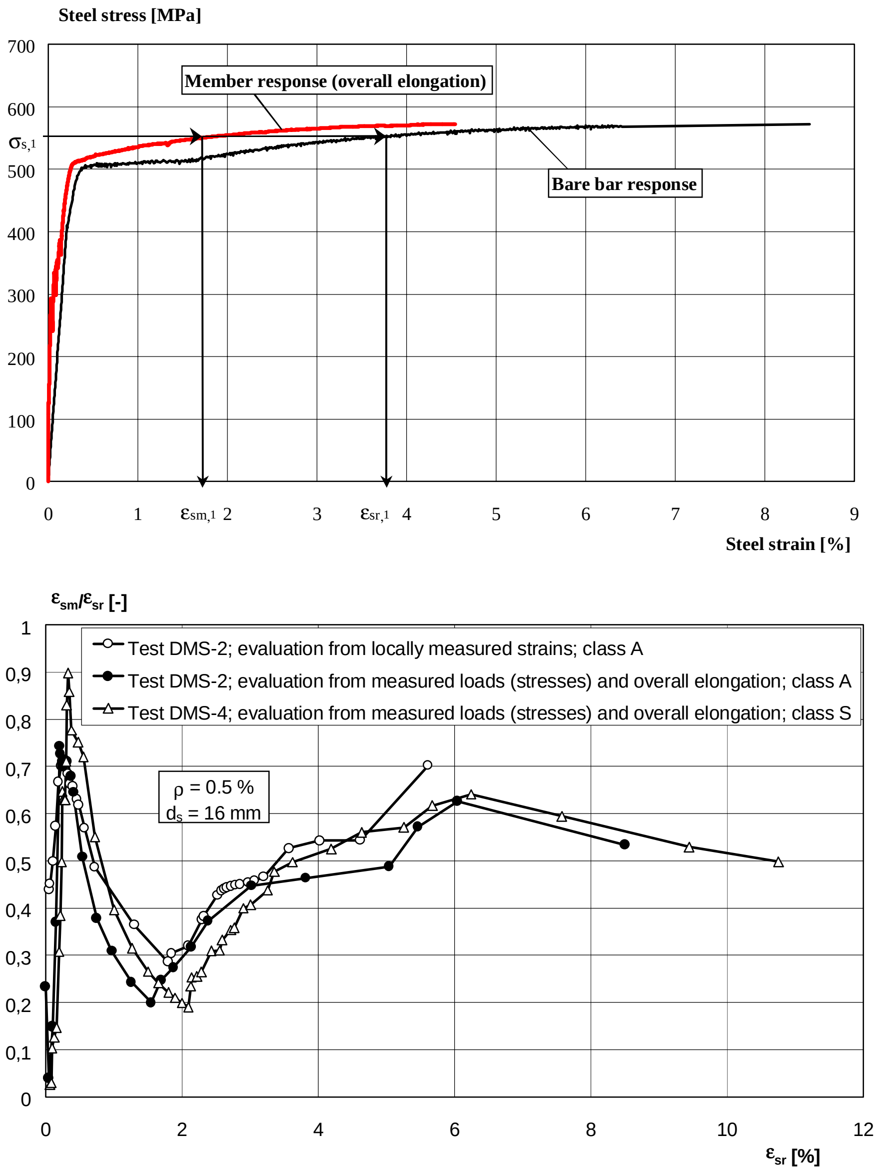


Fig. 3: Steel strain as a function of the distance from the pre-formed crack (r = 0.5%, B 25) for different load stages

After the strain at the crack reaches the yield plateau a strong localisation of strain in the vicinity of the crack takes place up to the load level that corresponds with strain hardening (strain at the crack ε_{sr} = 0.88 to 2.30%). With further increasing load (strain at the crack ε_{sr} = 3.57 to 5.62%) the plastified zone starts to extend and the strain gradient in the vicinity of the pre-formed crack decreases.



a)

b)

Fig. 4: Test results of tension members (ρ = 0.5% and bar diameter d_s = 16 mm): a) Stress-strain behaviour, b) Ratio $\varepsilon_{sm}/\varepsilon_{sr}$ as a function of ε_{sr}

These effects clearly appear if the ratio ε_{sm} / ε_{sr} is plotted as a function of ε_{sr}. Two methods of evaluating ε_{sm} / ε_{sr} are adopted. Firstly, the overall elongation and the measured loads (stresses) are used, as shown in Fig. 4a. Secondly, the locally measured strains are averaged over 26.5 to 259 mm distance from the pre-formed crack to get ε_{sm} and local strain measured at the crack ε_{sr} is employed. Fig. 4b shows the results of both evaluations and test results in similar case for steel class S evaluated according to the first method. Since the results of both evaluation methods agree reasonably well in the whole steel strain range, the method adopting loads and overall elongation can be reliably used in case of tests without strain gauges.

The general behaviour in the post yield range of steel can be described as follows: after passing the yield stress f_y the ratio e_{sm}/e_{sr} significantly reduces. The smallest values of e_{sm}/e_{sr} are reached when the steel strain at the crack is equal to the strain at strain hardening $e_{sr} \approx e_{sh}$ ($e_{sm}/e_{sr} \approx 0.20$). Hereafter with increasing strain at the crack the values of e_{sm}/e_{sr} start to increase and for high plastic strains $e_{sr} \approx 6\%$ achieve their maximum ($e_{sm}/e_{sr} \approx 0.60$). Then, for increased steel strains at the crack the ratio e_{sm}/e_{sr} decreases again. The use of steel classes A and S in this case (r = 0.5%) had no significant influence on the ratio e_{sm}/e_{sr} except for the slight difference in this ratio for strains at the crack $e_{sr} = e_{sh}$.

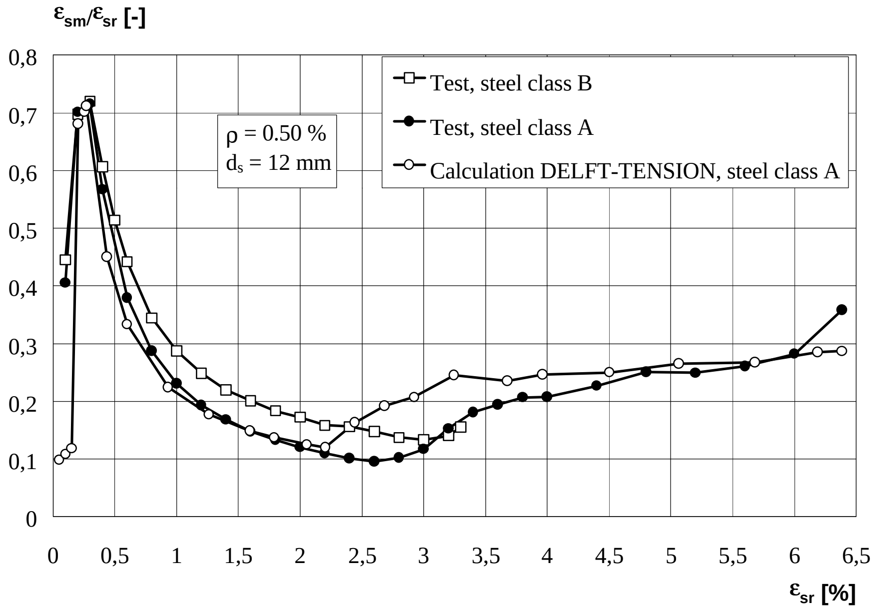


Fig. 5: Ratio between mean steel strain and steel strain at the crack e_{sm}/e_{sr} as a function of steel strain at the crack e_{sr} (r = 0.5% , bar diameter d_s = 12 mm). Influence of steel ductility and comparison with calculated results

The influence of using cold worked (steel class B without yield plateau) is shown in Fig. 5., where comparison is made with the case where hot rolled reinforcing steel (steel class A with a distinct yield plateau) is used (r = 0.5% and a bar diameter d_s = 12 mm). Up to steel strains at the crack $e_{sr} \approx 0.5\%$ there is no significant influence of steel ductility on the ratio e_{sm}/e_{sr}. With increasing strain at the crack the members reinforced with steel A have lower values e_{sm}/e_{sr} than these reinforced with cold worked bars ($De_{sm}/e_{sr} \approx 0.05$). The minimum ratio $e_{sm}/e_{sr} = 0.10$ for steel A is reached at $e_{sr} \approx e_{sh} \approx 2.5\%$. While for larger strains at the crack ($e_{sr} > e_{sh}$) the ratio e_{sm}/e_{sr} starts to increase again up to values 0.35 at ultimate strain when employing high ductile steel A. For low ductile steel B the ratio e_{sm}/e_{sr} drops down to $e_{sm}/e_{sr} \approx 0.13$ and practically increases no more until the ultimate strain at the crack ($e_{sr} \approx 3.3\%$) is reached. Comparing the results for different bar diameter (Fig. 4b and 5) a large difference for higher plastic steel strains at the crack appears and higher ratios e_{sm} / e_{sr} are measured due to the pronounced longitudinal cracking for the larger bar diameter. While the contribution of concrete between cracks in the range of inelastic steel strains is significantly influenced by the reinforcement ratio, bond failure mode and the steel ductility, the concrete strength has only small influence on the ratio e_{sm} / e_{sr}. The measured results for steel class A are compared with results calculated with DELFT-TENSION (Bigaj-van Vliet, 1998, based on Den Uijl; Bigaj, 1996). The agreement between measurement and simulation is very good in the whole steel strain range.

3. CONCLUSIONS

The basic requirements on bond between concrete and reinforcement in RC members are partly contradictory and cannot be fulfilled simultaneous. Currently produced deformed bars ensure good bond behaviour in the serviceabilty limit state and high capacity of anchorages and lap splices. However, at inelastic steel strains the contribution of concrete between cracks is likely to be too high. To evaluate the contribution of concrete at inelastic steel strains tests on RC columns loaded in uniaxial tension up to steel rupture were performed. The results indicate that the behaviour of RC elements beyond yielding is significantly influenced by steel ductility and reinforcement percentage. Significant differences are found for hot rolled and cold worked steel type, that are not captured by MC 90. Also the mode of bond failure (splitting or pull-out) strongly influences tension stiffening. At the current stage a rational bond model that accounts for yielding of steel is improved and, after incorporating the effect of bar surface geometry (f_R), it will be implemented in a non-linear FE-program. Optimisation of rib pattern of reinforcement in the view of the fundamental requirements on bond is aimed, using simulation results of RC members.

4. REFERENCES

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