STUDY OF THE TRANSFER OF TENSILE FORCES BY BOND IN ECCENTRIC REINFORCED CONCRETE MEMBERS

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ABSTRACT

The bond between concrete and steel is of fundamental importance to deformation characteristics of concrete structures. The phenomenon of force transfer from a steel bar to a surrounding concrete has been extensively studied from early seventies[1,2]. Many parameters, such as concrete cover [3], bar spacing, deformation pattern of a bar, bond length, concrete strength and quality, type of loading and so on [4,6] were taking into account during those investigations. There are a lot of experimental results, but most of them were obtained from test of very short bond length and relatively large covers. What else, they were mostly from uniaxial tension specimen. Analytical models, connected with those experiments, assume that the local bond stress τ_b depends only on the relative local slip Δ and is independent of a distance from loading end or from cracked cross section[5].

Experimental investigations were performed on eccentric tension reinforced concrete members of a lengths of more than 50 bar diameter. Such a length allows to arise of several primary cracks and some internal ones. The specimen shape allowed to use electric strain gauges and elastooptical surfaces to observe strain changes along a steel bar. The results proved that uniaxial tension test is not suitable to establish a bond phenomenon of members under bending. Typical relationships between a bond stress and a slip is also not valid. Proposed specimen and analytical model seem to be more appropriate.

EXPERIMENTAL INVESTIGATION

Typical specimen used in the main test is shown on fig.1. A half of it is covered by an elastooptical surface. Strain gages were glued on a bar one next to another. Their number varied for different bars from 19 to 21. Strain gages on concrete were mixed in three cross sections. In a tension zone they were one to another and on the convex part they were mixed in 30^0 distance. Total elongation of the specimen and width of cracks were measured using displacement transducers. The material used in the preparation of concrete were ordinary portland cement, natural sand and graded gravel of 8 mm maximum size with water to cement ratio of 0.5. An average concrete strength (cube 150 mm) was about 24 MPa, a tension splitting strength 2.5 MPa and modulus of elasticity for a first monotonic loading – 31 GPa. After a day from casting all specimen were storage in a water for a week. Steel bars were hot rolled steel type 34GS. They were moulded to its half and their areas were calculated on a base of their weight.

In the main test eight specimen were used. Those were double pull out test with a force control. They were loading up to very near of yielding of a bar and than unloaded to a little above zero. Than there were several cyclic loading for each specimen and later they were tested to failure. In all cases test were finished when strain in steel has been increasing while a force stared to decreasing

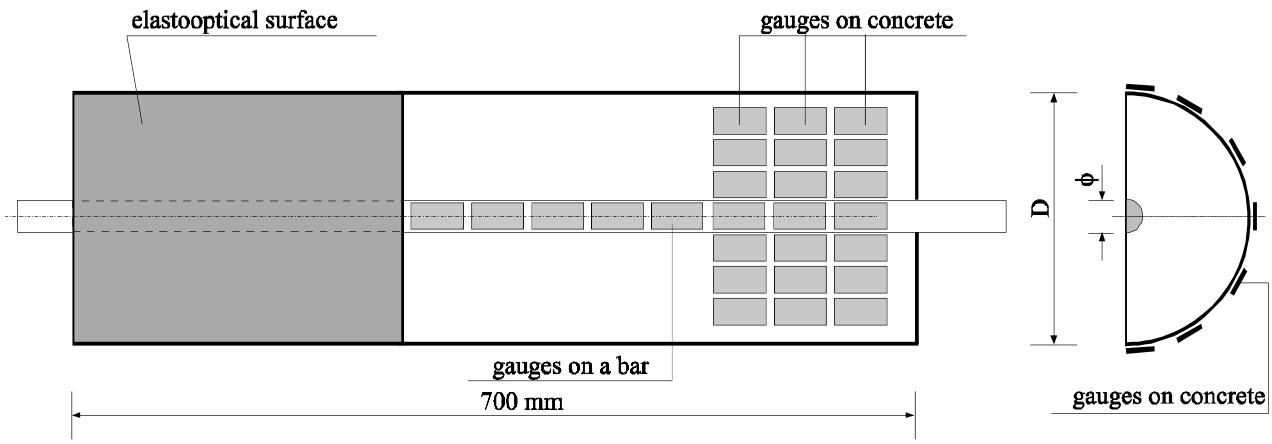

Fig.1.Typical specimen used for experimental researches.

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ANALITICAL MODEL

The main assumptions were presented earlier and are widely accepted. The governing equations are as follows:

a) equilibrium conditions for the whole cross section,

b) relationship between strains and a slip $\frac{d\Delta}{dx} = \varepsilon_s(x) - \varepsilon_c(x)$

c) equilibrium condition of bond stress and a steel stress $\frac{d\sigma_s(x)}{dx} = -\frac{4\tau_b(x)}{d_b}$,

d) bond function,

The most important differences are formulae for a bond stress. Nowadays, the most popular relationships are based on results of experimental researches from pull out test with a very short bond length and rather large concrete cover. It is assumed that in spite of this length, bond stress is constant and can be calculated from Eqn. 1 and ascending branch of the relationship (fig.2.) follows the Eqn.2.

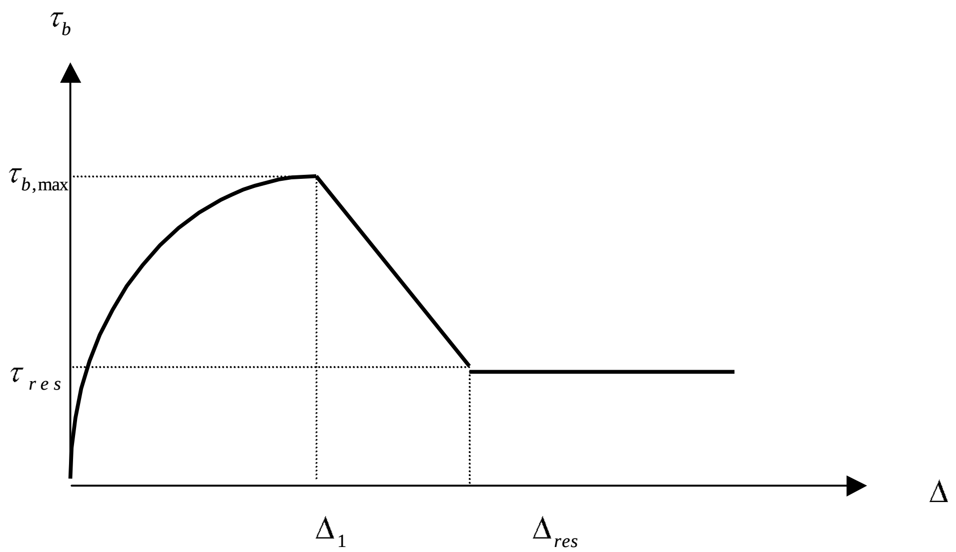

Fig.2.Typical relationship between a slip **Δ** and a bond stress τ_b.

$$\tau_b = \frac{F}{\pi d_b l_b} \quad , \tag{1}$$

where F – tensile force, d_b – bar diameter, l_b – bond length. A slip Δ is usually measured at a free end or is taken as an average.
It is also assumed that the relationship in Eqn.2 is unique, i.e. is independent of a distance from an end of specimen or a cracked cross section. Such a method leads to some mistakes. Very short bond length makes internal cracks impossible to arise. Values of bond stress obtained from Eqn.1 are unrealistic – very large. For the sake of large covers, splitting failure is also excluded. The maximum value of bond is at cracked cross section, where there is no bond. The rate of change of a steel stress is the greatest very near a crack. Some researches [7] , where a steel strain was measured, show that strain distribution along a bar has an intersection point (fig.3.) while Eqn.2 leads to curve similar to that dotted one .

$$\tau_b = \tau_{b,\max}\left(\frac{\Delta}{\Delta_1}\right)^{\alpha} \tag{2}$$

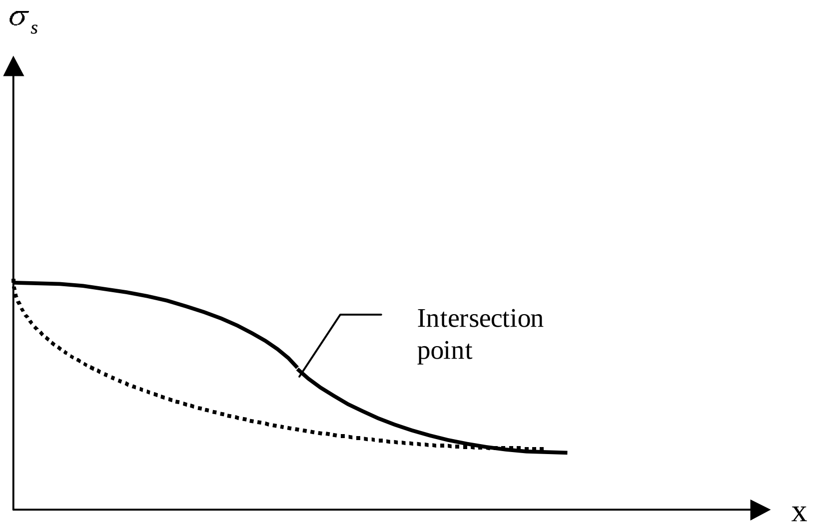

Fig. 3. Steel stress distribution along a bar – from eq.(2) -dotted line and directly from experiments.

The way of a transfer of a force from a steel bar to concrete, in pull out test, depend on a level of loading. At the beginning, in most cross section there is still perfect bond and strains in steel and concrete are $\varepsilon_s = \varepsilon_c$. Only for cross section next to ends, there is a relative slip between steel and concrete. When the load increases, the distance of a broken original bond also increases but in a rest of a specimen (particularly in the middle part) still $\varepsilon_s = \varepsilon_c$. This situation lasts until the first crack arises in a middle part of a member. At this moment, original bond is also broken next to the crack. For this level of loading bond stress can be described by Eqn.3.

$$\tau_b(x) = k(a - x)\sqrt{x}\,, \tag{3}$$

where ***x*** is a distance from a cracked cross section , ***k*** and ***a*** bond parameters.
This relationship is valid only for cracking level of loading and it is used to determinate the bond transfer length $l_b = a$. According to [8] , it can be expressed as:

$$a = l_b = \frac{15}{3^{1.5}} \frac{f_{ctm}}{\tau_{b,\max}} \frac{\left(\frac{3\pi^2}{32} - \frac{2}{3}\right)}{\pi\rho^2} d_b\,, \tag{4}$$

where f_{ctm} - average tensile strength of concrete, $\rho = \frac{d_b^2}{D^2}$ and D is specimen diameter.

Equation (4) has a form due to a test specimen shape. It is worth to notice that the most important bond parameter is $\beta = \frac{f_{ctm}}{\tau_{b,\max}}$ and it is established experimentally for cracking level of loading.

When the load is a bit greater, the bond function is assumed to be:

$$\tau_b(x) = gx\sigma_s(x)\,, \tag{5}$$

where ***g*** is bond parameter. Equation (4) leads to the next formulae for a steel stress distribution:

$$\sigma_s(x) = \sigma_0 e^{-\frac{2gx^2}{d_b}}\,, \tag{6}$$

where σ_0 is a stress in a steel bar at cracked cross section.

At this level of loading , $\tau_{b,\max}$ remains constant. The peak value is located at point $x_{\max} = \sqrt{\frac{d_b}{4g}}$ and its value equals $\tau_{b,\max} = \sigma_0\sqrt{\frac{d_b g}{4e}}$. When load (and steel stress) increases, the bond parameter ***g*** must decreases. It implicates the movement of the peak value location away from cracked cross section - $x_{\max}$ increases. Some experimental researches confirm this phenomenon. It takes place when only microcracking exist close to a steel bar. When peak value reaches the point ***a*** , the bond parameter ***g*** becomes constant and the maximum bond stress starts to increase. At the same time, internal cracks caused by both tension forces and additional elongation from bond forces, start to develop. When bond stress is relatively large and a concrete cover is rather small, the internal cracks can reach the bottom side of a member. Than they act in a similar way as the primary cracks but their width is significantly smaller. The model proposed by Tepfers can describe this process quite well. In same cases, the internal cracks remain inside a concrete mass until the failure of a steel bar.

RESULTS

The specific shape of specimen allowed to look after many interesting phenomenon. In this paper special regards were taken to a steel stress and a bond stress distribution along a steel bar in a member under eccentric tension. Two models of a bond function were compered. Typical steel stress distribution is shown on a figure 4. The specimen has diameter 100 mm and is reinforced with a bar of 12 mm diameter. There are five primary cracks but two of them are below an optical surface. After the first monotonic loading up to a level very near to yielding of a steel, the specimen was cyclic loaded and then loaded to a failure. Similar to other specimen, the failure was caused by yielding of a steel.

Specimen Behavior Before Cracking

At the begging, a steel distribution along a bar was almost uniform except for some local disturbance. They were caused by non homogeneity of concrete, shrinkage and local deformation of a steel bar. Soon, for tensile force slightly above 1 kN, they vanished. Up to a level 4 kN, steel distribution remained uniform except the loaded end. There, from early begging, the steel strain was higher than the concrete strain. On the optical surface, it was possible to observe the strain concentration in concrete at those places. It had a triangular shape with a base in an edge. When load was increasing, the base and the extent of that triangle

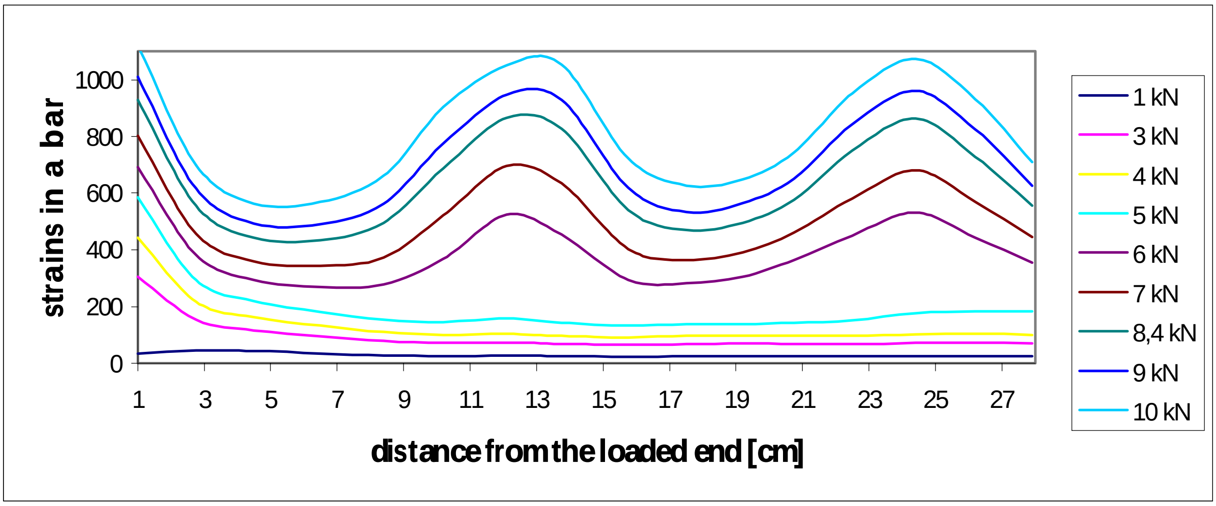

Fig.4. Steel strain distribution along a bar (for specimen No. 2)

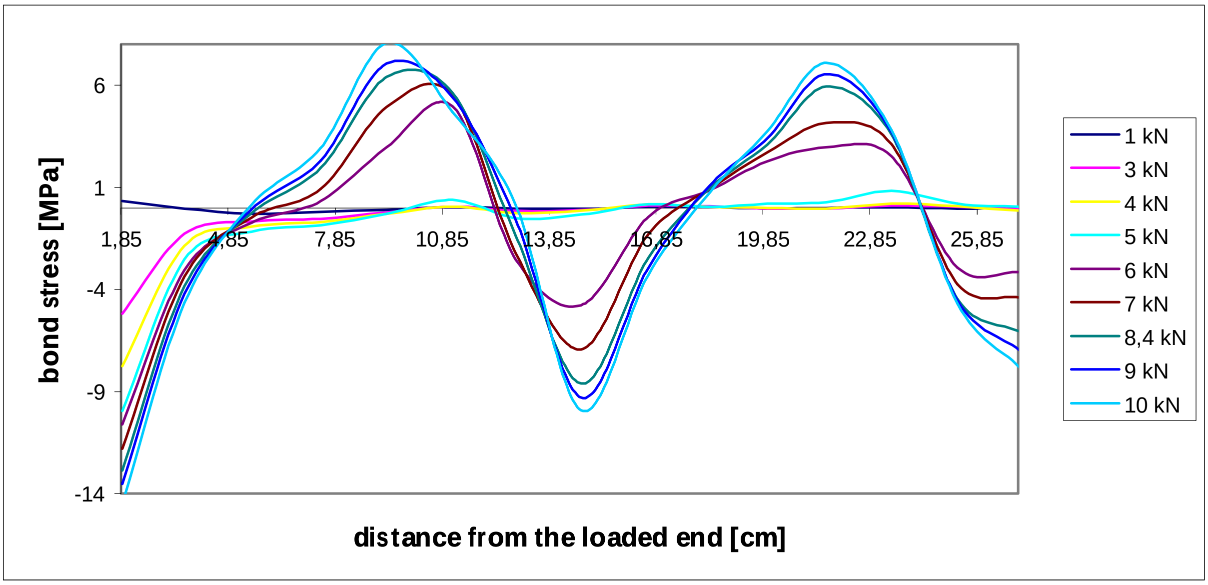

Fig.5. Bond stress distribution near a loaded end and near to cracks.

also increased. For other cross section some differences in strain distribution could be observed for forces 4-5 kN. They were very small and were caused by local weakness of a concrete. At two points, 124 mm and 238 mm, they turned to cracks.

Even before cracking, it was easy to notice that ended parts of a member behave in a different manner than the middle part of a member. Local disturbances at loaded ends can be easily explained. Those cross sections act as they near a crack even for very low level of loading. From early begging, $\varepsilon_s > \varepsilon_c$ and there is a slip Δ. It is strictly connected with strain concentration which reaches a distance about 4 to 5 d_b.

Cracking Level Of Loading

In all cases, the first crack appeared very close to the middle of the specimen. The next cracks – when load was only a bit greater or at the same time. The first crack formed at the edge of a concrete and developed toward a steel bar. The next cracks could start both from a bar and an edge of concrete. Cracks, which were next to loaded ends, appeared later. This phenomenon is easy to explain by an influence of a slip existing already before cracking [8]. Spacing between cracks were very close to ***a*** value (for specimen No. 2 ***a*** = 89 mm and cracks spacing – from 96 mm to 110 mm) . Only for cracks near loaded ends, the spacing was sufficiently greater – 125 – 130 mm. Their width was smaller than others.

Steel Strain Distribution Along A Bar After Cracking

Steel strain distribution along a steel bar is shown on the fig.4 and a bond stress distribution – on fig 5. Bond stresses were calculated using eqn. 1. From these figures, it is easy to observe that a shape of curves is different near a loaded ends and near a cracks. In the table 1 and 2 , results of verification of Eqn. 2 are presented. Bond slip relationship was taken as $\tau_b = k\Delta^{\alpha}$. The slip was calculated approximately as a steel strain multiplied by a distance between the place where the bond stress equals zero and a chosen cross section.

TABLE 1
VERIFICATION OF BOND SLIP FUNCTION

Distance from the loaded end	18 .75mm		36.25 mm		55 mm	
	Δ [μm]	τ_b [MPa]	Δ [μm]	τ_b [MPa]	Δ [μm]	τ_b [MPa]
1	27.5	14.5	14.0	4.7	2.8	2.2
2	24.3	13.5	12.4	4.2	2.5	2.1
3	21.8	12.9	10.9	3.8	2.2	2.05
4	18.0	11.8	8.9	3.4	1.8	1.9
5	14.8	10.6	7.2	3.1	1.4	1.9
6	11.4	9.9	5.3	2.5	1.0	2.1
7	8.4	7.7	3.8	1.9	0.7	1.4
k	0.012		2.137		0.060	
α	1.956		1.506		2.570	
R^2	98.17		99.22		59.05	

In the Table1, the cross section near the loaded end is taken into account and in the Table 2 – the cross section near a crack located at x = 33.8 cm from the end. From Table 1, it is worth to notice the excellent agreement between experimental values and a shape of theoretical curve for cross section close to the end. The same can be observed for cross sections near a crack. This explains why the function of bond slip from Eqn. 1 is so popular. At the other hand, predicted values of parameters ***k*** and ***α*** differ very much for different position of cross sections. It confirms the observation that bond slip function is not unique. May be a more exact results could be obtained if the function has the next form:

$$\tau_b(x) = k\Delta^{\alpha}\Psi(x), \tag{7}$$

where $\Psi(x)$ could be a simple function of the distance from the crack.

Table 3 shows results of comparison between some experimental values and ones obtained from theoretical model presented in that paper. It confirms the main tendencies (moving a peak value – $x_{\max}$ and decreasing value of ***g***) but there are also some important differences. The assumption of constant value of $\tau_{b,\max}$ is only simplification. In fact, during loading both $x_{\max}$ and $\tau_{b,\max}$ increase until $x_{\max}$ reaches the distance ***a.*** This observation must be taken into account in the improved model of force transfer in reinforced concrete members under eccentric tension or bending.

TABLE 2

VERIFICATION OF BOND SLIP FUNCTION

Distance from the loaded end	234.5 mm		216.25 mm		199.75 mm	
	Δ [μm]	τ_b [MPa]	Δ [μm]	τ_b [MPa]	Δ [μm]	τ_b [MPa]
1	46.1	3.8	29.1	7.1	16.2	3.7
2	40.4	3.7	25.3	6.5	14.0	3.4
3	35.8	3.6	22.3	5.9	12.3	3.1
4	28.1	3.1	17.6	4.1	9.7	2.7
5	22.0	2.5	13.8	2.9	7.6	2.3
k	34.8		58.4		5.0	
α	1.645		0.788		1.592	
R^2	94.28		97.78		99,93	

TABLE 3

VERIFICATION OF BOND FUNCTION

	From experiment after smoothing					From theoretical model				
	1	2	3	4	5	1	2	3	4	5
g [cm^{-1}]	0.0172	0.0191	0.0204	0.0209	0.0207	0.0083	0.010	0.0128	0.0206	0.0339
$x_{\max}$ [cm]	4.17	3.97	3.83	3.79	3.80	6.00	5.38	4.83	3.81	2.97
$\tau_{b,\max}$ [MPa]	9.35	8.82	8.19	6.53	5.07	6.5				

CONCLUSIONS

a) Longitudinal cracks ,caused by splitting forces from bond, do not lead to bond failure in eccentric loaded reinforced concrete members. This is one of the most important difference between that kind of members and uniaxial tension specimen.

b) Longitudinal cracks are more dangerous than flexural ones for the sake of corrosion of a bar. This means that they can not be acceptable for service loading.

c) Typical specimen of the short bond length and relatively large concrete cover are very suitable for resting but they miss some important phenomenon existing in a real much longer member.

d)Well known relationship $\tau_b = \tau_{b,\max}\left(\frac{\Delta}{\Delta_{\max}}\right)^{\alpha}$ is base on results from a very short bond length specimen and it not proper for large members under bending or eccentric tension. Besides, it can be used only in numerical calculations for it leads to a non-linear differential equation which has not general analytical solution

e) It is reasonable to assume that the proper bond function has a general form like $\tau_b(x) = k\Delta^{\alpha}\Psi(x)$ or similar.

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