

Influence of loading condition and reinforcement size on the concrete/reinforcement bond strength

Kazim Turk†

Department of Civil Engineering, Engineering Faculty, Firat University, Elazig, Turkey

Sinan Caliskan†

*Division of Civil Engineering, Faculty of Engineering and Physical Sciences,
University of Dundee, Dundee, U.K.*

M. Sukru Yildirim‡

Department of Civil Engineering, Corlu Engineering Faculty, Trakya University, Edirne, Turkey

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Abstract. The paper reports on a study of bond strength between reduced-water-content concrete and tensile reinforcement in spliced mode. Three different diameters (12, 16 and 22 mm) of tensile steel were spliced in the constant moment zone, where there were two bars of same size in tension. For each diameter of reinforcement, a total of nine beams ($1900 \times 270 \times 180$ mm) were tested, of which three beams were with no axial force (positive bending) and the other six beams were with axial force (combined bending). The splice length was selected so that bars would fail in bond, splitting the concrete cover in the splice region, before reaching the yield point. It was found that there was a considerable size effect in the experimental results, i.e., as the diameter of the reinforcement reduced the bond strength and the deflection recorded at the midspan increased significantly, whilst the stiffness of the beams reduced. It was also found for all reinforcement sizes that higher bond strength and stiffness were obtained for beams tested in combined bending than that of the beams tested in positive bending only.

Key words: bond strength; tensile reinforcement; size effect; lap splice; combined bending; load-deflection stiffness.

1. Introduction

Interfacial bond strength between steel and the surrounding concrete is an important factor influencing strength and durability of reinforced concrete structures. The term bond is defined as the resistance against slip between steel and concrete and it is essentially composed of three components: chemical adhesion, friction and mechanical interaction between the ribs of the bar and concrete (Hamad and Mansour 1996). Nevertheless, it was found that the reinforcing bar diameter

† Doctor, Corresponding author, E-mail: kturk@firat.edu.tr

‡ Assistant Professor

had very important effect on the bond strength (Turk and Yildirim 2003). De Larrard *et al.* (1993), reported that high strength concrete provided about 80% higher bond strength with 10 mm diameter reinforcement steel than that of normal strength concrete with same size reinforcement, whilst with 25 mm diameter steel reinforcement, the difference dropped to 30% indicating the influence of reinforcement size and concrete quality on the bond strength.

Because of limitations of bar lengths available, requirements at construction joints, and changes from larger diameter bars to smaller bars, splices of reinforcing bars are often necessary. The most common method of splicing is simply to lap the bars one over the other. Lapped bars may be either spaced from each other or placed in contact. However, the contact splices are much preferred since the bars can be wired together (Chaallal and Benmokrane 1993, Tighiouart *et al.* 1998). It is noted that splice behaviour is influenced by splitting cracks developing along the bars (Tepfers 1973) and by flexural cracks that mainly develop at the splice ends (Giovanni *et al.* 1996). Both types of crack are essentially governed by the interfacial bond strength between steel reinforcement and concrete. Of which the flexural cracks, in particular, are closely related to the maximum slip of steel, which depends on the local micro crushing of the porous concrete layer in front of rib (Gambarova and Giuriani 1985, Giuriani 1981). Splitting cracks, on the other hand, are caused by rib-wedging action and are governed by the bond strength and stiffness of concrete (Tassio and Koroneos 1984, Tepfers 1979).

The aim of this paper is to investigate the influence of reinforcement size and different loading conditions on the bond strength between concrete and tensile reinforcement in splice region. Furthermore, a comparison between the results obtained from this study and that of Orangun *et al.* (1977, 1975), Esfahani and Rangan (1998) and Darwin *et al.* (1996) is made.

2. Experimental program

A total of twenty seven geometrically identical beam specimens made of 30 MPa reduced-water-content concrete were tested to determine the bond strength between tension reinforcement and concrete. A water to cement (w/c) ratio of 0.33 was used for concrete mix, whilst the required workability was attained using superplasticizer. The geometrical details and compressive strength

Table 1 Details of test specimens

Beams	d_b (mm)	l_s (mm)	N (kN)	Number of beams tested	f'_c (MPa)	ρ (A_b/bd)
B12	12	235	0	3	30.4	0.0054
			7.5	3	32.1	0.0054
			15	3	31.1	0.0054
B16	16	235	0	3	30.1	0.0095
			8.5	3	32	0.0095
			17	3	31	0.0095
B22	22	235	0	3	30.5	0.0180
			10	3	30	0.0180
			20	3	30.4	0.0180

Table 2 Mix proportions (kg/m^3) used to prepare the beam specimens

Portland cement	Water	Sand (0-5 mm)	Gravel (5-8 mm)	Superplasticizer
350	115.50	1320	566	7

Table 3 Properties of steel reinforcement used

d_b (mm)	A_b (mm^2)	f_y (MPa)	f_{su} (MPa)	Elongation percent
12	113.10	476.48	719.97	17.23
16	201.06	454.63	671.63	19.44
22	380.13	446.13	663.18	20.97

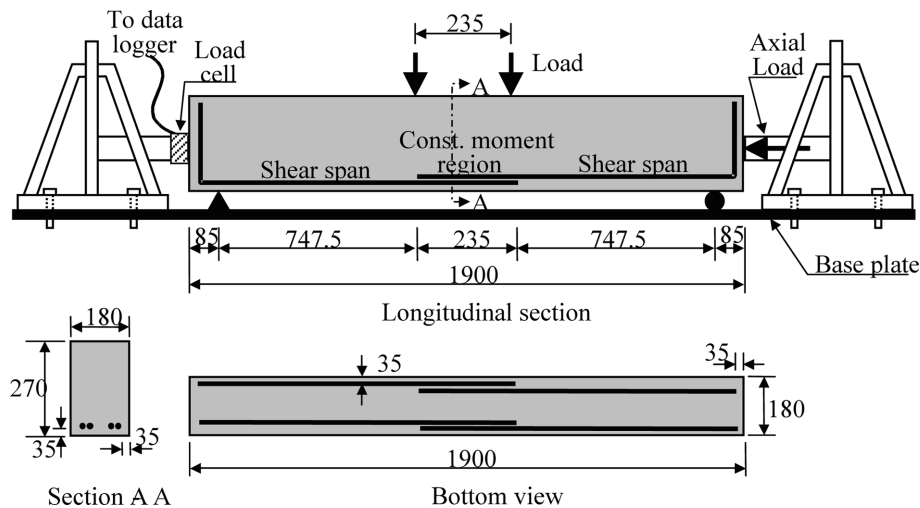


Fig. 1 Loading arrangement and geometrical details of beam specimen (All dimensions are in mm)

values of concretes at 28 days are given in Table 1 and the mix proportions are given in Table 2. Three different diameters (12, 16 and 22 mm) of steel reinforcement have been selected, the mechanical properties of which are shown in Table 3. Each beam contained two-tension reinforcement as shown in Fig. 1 and a total of three beams for each diameter were tested.

The lap-spliced tension reinforcement was located in the midspan in a region of constant positive bending as shown in Fig. 1. The splice length, l_s was selected so that the bars would fail in bond, splitting the concrete cover in the splice region before reaching the yield point.

Test beams were cast in a horizontal position with the lap spliced bars located at the bottom of the steel mould. Poker vibrator was used to attain optimum compaction. Following casting, the specimens were covered with wet burplap, which continued for 28 days following de-moulding the specimens after 24 hours. All specimens were tested at 28 days.

3. Test set-up and procedures

The test set-up and the four point loading arrangement used during the load controlled experiments are given in Fig. 1. Beams were simply supported over a span of 1730 mm and tested until failure takes place. An incremental load of 0.0002 N/sec was applied through a 5000 kN capacity testing machine. The load from testing machine was transferred through a stiff steel girder onto the specimens in the form of two equally concentrated loads. The pre-defined axial load was reached prior to starting experiments. This load was kept at this level throughout the test until failure takes place.

The load and the displacement at the centre of the specimens were continuously recorded throughout the tests. Cracks at the side and bottom faces of the specimens were marked for further analysis. Concrete cover over the splice length in all specimens was first to fail due to the interfacial bond failure between reinforcement bars and concrete.

4. Mode of failure

The first flexural cracks in all beams appeared randomly in the constant moment region on the tension side of the beams outside the splice. As loading progressed, cracks were formed along the entire length of the constant moment region including the splice. In all specimens, failure diminishing of load carrying capacity took place at maximum load just after the longitudinal splitting cracks started to form along the splices. The longitudinal cracks were formed at the bottom face adjacent to the reinforcement bars.

Typical load-deflection curves for the beams obtained from the tests are given in Fig. 2. It is shown that the maximum load decreased as the diameter of the reinforcement bar was reduced, whilst the deflection recorded at the centre of the beam increased. The crack pattern shown in Fig. 3 was similar for all specimens. However, smaller crack widths were observed on the specimens containing smaller diameter steel reinforcement. In addition, more cracks were formed in beams tested in combined bending (Fig. 3a) than that tested in positive bending only (Fig. 3b).

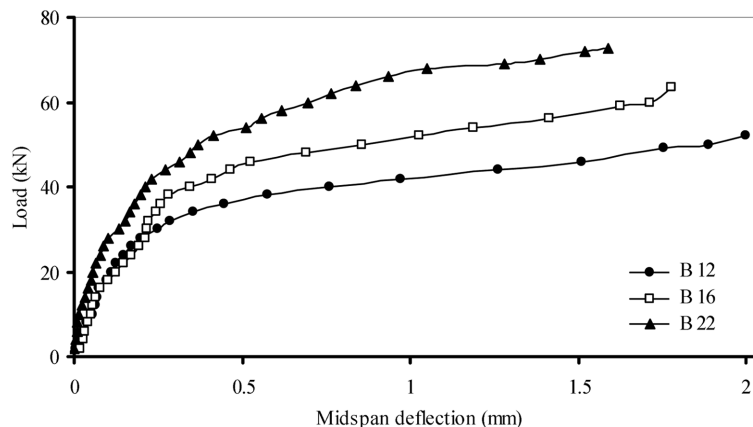


Fig. 2 Typical load-deflection curves for beams ($N = 0$)

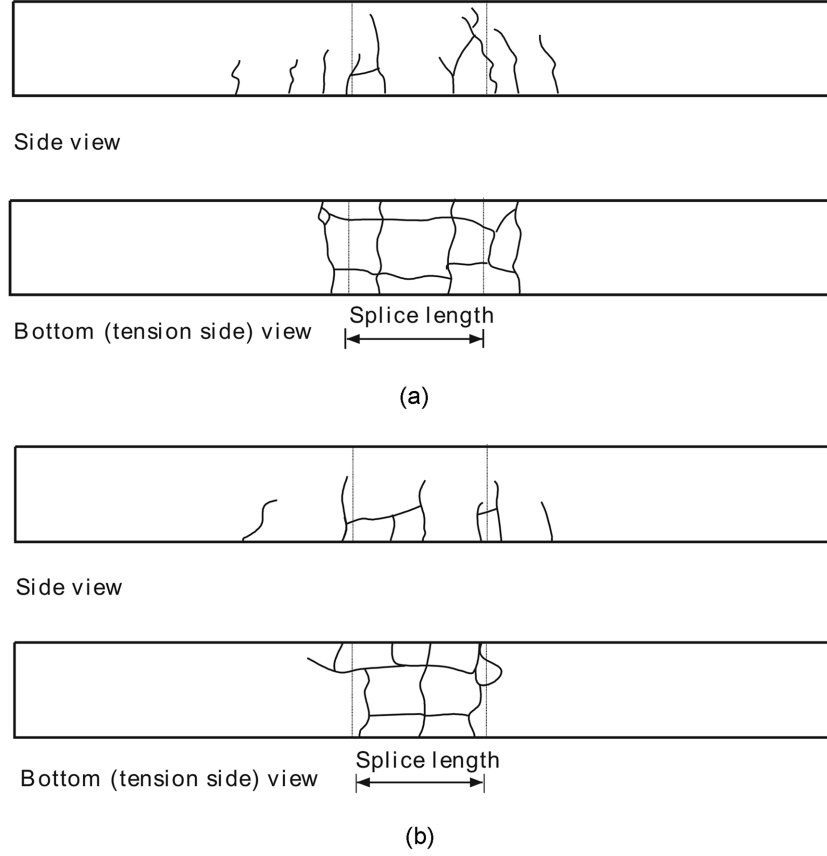


Fig. 3 Typical crack pattern obtained from B12, (a) $N = 7.5$ and (b) $N = 0$

5. Analysis of results

The failure mode in all specimens was a face-and-side split failure suggesting that the splice reached its maximum capacity. Therefore, bond strength could be determined via the stress developed in the steel, f_s , which was calculated using elastic cracked section analysis and was determined from the maximum load obtained for each beam specimen. In this analysis the modulus of elasticity of steel, E_s was taken as 203,000 MPa and of concrete, E_c was obtained by $E_c = 4730\sqrt{f'_c}$ MPa (ACI 318-89). During the analysis, tensile stress in concrete below the neutral axis was not taken into account and linear stress-strain behaviour was assumed. To obtain the average bond stress, u_t the total force developed in the steel bar ($A_b f_s$, where A_b is the cross-sectional area of the bar) was divided by the surface area of the bar over the splice length ($\pi d_b l_s$) as follows:

$$u_t = \frac{(A_b f_s)}{\pi d_b l_s}; \quad u_t = \frac{f_s d_b}{4 l_s} \quad (1)$$

where d_b is the bar diameter and l_s is the splice length. Neutral axis width (x) and steel stress (f_s) are given in Table 4.

Maximum load and average bond strength values (u_t) for each reinforcement diameter are given in

Table 4 Experimental results (average of three beams) with three different sizes of bar

Specimens	N (kN)	e (mm)	x (mm)	f_s (Mpa)
B12	0	-	58.68	394.38
	7.5	2756	60.00	408.02
	15	1465	62.24	417.84
B16	0	-	74.85	273.95
	8.5	2981	76.01	289.67
	17	1643	78.15	310.34
B22	0	-	95.69	175.75
	10	2979	98.40	187.30
	20	1643	100.04	201.17

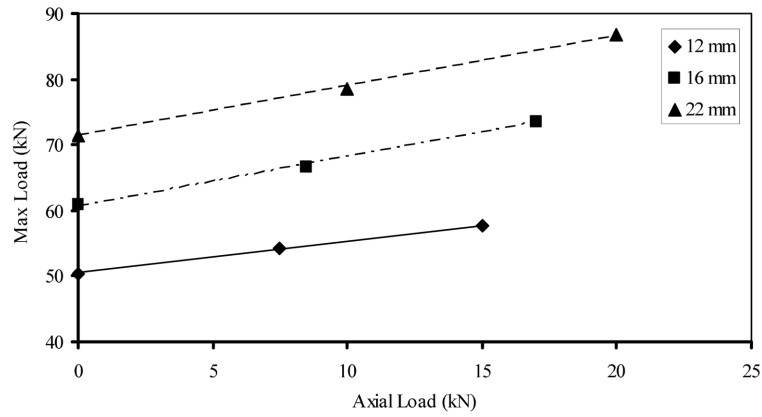


Fig. 4 Maximum load applied to beams with various sizes of reinforcement

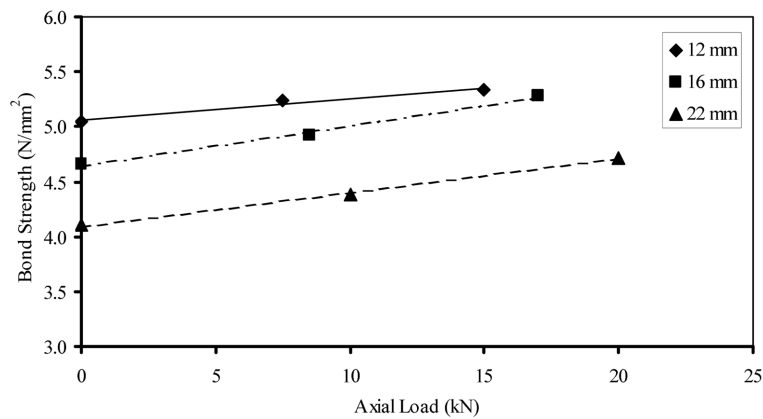


Fig. 5 Bond strength results of beams with various sizes of reinforcement

Figs. 4 and 5, respectively. It is shown that as the size of the reinforcement decreases the bond strength between reinforcement and concrete increases significantly indicating a clear size effect,

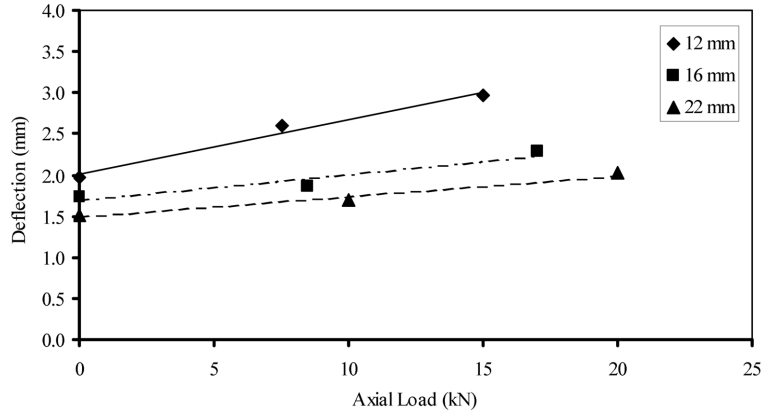


Fig. 6 Deflections at P_{\max} at the centre of beams with various sizes of reinforcement

i.e., the smaller the diameter of the reinforcement the higher the bond strength between concrete and the steel bar. It is also indicated in Fig. 5 that as the axial force increased the bond strength increased regardless of the size of reinforcement. However, the increase in bond strength due to the axial force was not proportional to the increase in axial force.

The deflection values recorded at P_{\max} are given in Fig. 6. It is shown that deflection reduces as the diameter of the reinforcement increases indicating that as the diameter of the steel reinforcement increases the stiffness (resistance to deflection) of the beams increases. Furthermore, it is also shown that when the beams were subjected to combined bending, i.e., with axial force, bond strength and stiffness of the beams increased irrespective of the diameter of the reinforcement compared to the beams tested in positive bending only.

6. Comparison with other researchers

Experimental bond strength results of the beams with three different diameters of reinforcement were compared to that of theoretically predicted values, by using the empirical equations developed by Orangun *et al.* (1977, 1975):

$$u = [1.2 + 3(c/d_b) + 50(d_b/l_s) + K_{tr}] \sqrt{f'_c} \quad (2)$$

where $K_{tr} = \frac{A_{tr} f_{yt}}{500s d_b}$, and by Esfahani and Rangan (1998):

$$U = u_c \frac{1 + 1/M}{1.85 + 0.024 \sqrt{M}} \left(0.88 + 0.12 \frac{c_{med}}{c_m} \right) \quad (3)$$

where $u_c = 4.9 \frac{c_m/d_b + 0.5}{c_m/d_b + 3.6} f_{ct}$ for $f'_c < 50$ MPa, $u_c = 4.9 \frac{c_m/d_b + 0.5}{c_m/d_b + 3.6} f_{ct}$ for $f'_c \geq 50$ MPa, and $M =$

$\cosh\left(0.0022 L_d \sqrt{R \frac{f'_c}{d_b}}\right)$, and U and f'_c are in MPa; $f_{ct} = 0.55 \sqrt{f'_c}$. c_m is the smallest value and c_{med}

is the second larger value of side cover, bottom cover or 1/2 of center-to-center spacing of bars. R varies between 3 and 4.25, which depends on type of reinforcing bar.

Moreover, experimental bond strength results of the beams were also compared to equation developed by Darwin *et al.* (1996):

$$\frac{T_c}{(f'_c)^{1/4}} = \frac{A_b f_s}{(f'_c)^{1/4}} = [63L_d(c_m + 0.5d_b) + 2130A_b] \left(0.1 \frac{c_M}{c_m} + 0.9 \right)$$

in which $(f'_c)^{1/4}$ is psi.

c_m, c_M : minimum and maximum value of c_s or c_b ($c_M/c_m \leq 3.5$), in.

c_s : $\min(c_{si} + 0.25\text{in.}, c_{so})$, in.

c_{si} : one-half of clear spacing between bars, in.

c_{so}, c_b : side cover and bottom cover of reinforcing bars, in.

Equation developed by Darwin *et al.* (1996) can be expressed in terms of u as:

$$u = \frac{(f'_c)^{1/4}}{d_b \pi L_d} [63l_s(c_m + 0.5d_b) + 2130A_b] \left(0.1 \frac{c_M}{c_m} + 0.9 \right) \quad (4)$$

The results are given in Table 5 and in Fig. 7. The measured bond stress for each specimen was divided by the predicted values to obtain the bond efficiencies listed in Table 5. The mean bond

Table 5 Bond efficiencies of Orangun *et al.* (1975, 1977), Esfahani and Rangan (1998) and Darwin *et al.* (1996), ($N = 0$)

Specimens	Measured bond stress, u_t (MPa)	Predicted bond stress (MPa)			Bond efficiency		
		Orangun <i>et al.</i>	Esfahani and Rangan	Darwin <i>et al.</i>	$u_t/u_{Orangun}$	$u_t/u_{Esfahani}$	u_t/u_{Darwin}
B 12	5.03	5.75	4.86	5.38	0.87	1.03	0.93
B 16	4.66	5.09	4.62	4.75	0.92	1.01	0.98
B 22	4.11	4.88	4.41	4.69	0.84	0.93	0.88

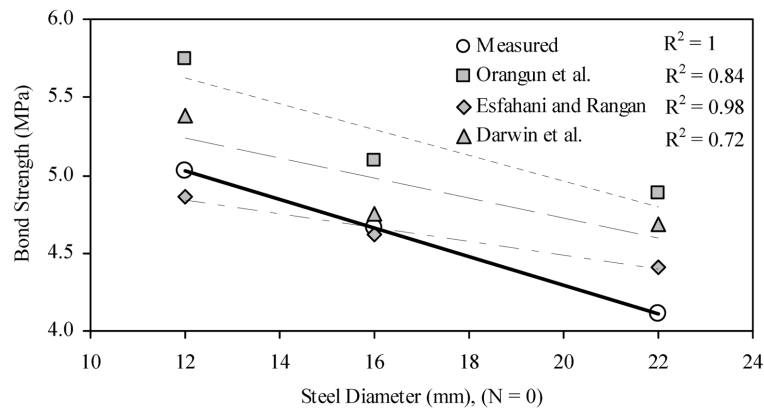


Fig. 7 Comparison of measured and the predicted bond strength results

efficiency for all bar splices using Eq. (2) of Orangun *et al.* (1977, 1975), is 0.88 with a standard deviation of 0.04. Moreover, Eq. (3) of Esfahani and Rangan (1998), is 0.99 with a standard deviation of 0.05 and Eq. (4) of Darwin *et al.* (1996) is 0.93 with a standard deviation 0.05. It is seen in Table 5 and in Fig. 7 that the predicted bond strength values by using Eq. (3) were closer to that of the experimental values. However, it is clear that the Eqs. (2) and (4) sometimes overestimates the bond strength between reinforcement and concrete.

Based on the experiments and interpretations in this study, the results indicate that the method developed by Esfahani and Rangan (1998) provides a better estimate of bond strength than that developed by Orangun *et al.* (1977, 1975) and Darwin *et al.* (1996).

7. Conclusions

Twenty-seven beam specimens containing an overlapping splice of two bars of same size (12, 16, and 22 mm in diameters) were used to investigate the effect of reinforcement size and loading condition on bond strength between steel and concrete. Based on the analysis of data, it is shown that there is a distinct size effect in the experimental results, i.e., as the diameter of the reinforcement reduces the bond strength and the deflection recorded at the centre of the beam increases significantly, whilst the load-deflection stiffness of beams reduces.

The bond strength and the load-deflection stiffness increased for the beam specimens subjected to combined bending compared to that tested in positive bending only, regardless of the size of reinforcement used. In addition, the experimental findings were compared to Orangun *et al.* (1977, 1975), Esfahani and Rangan (1998) and Darwin *et al.* (1996). The method developed by Esfahani and Rangan (1998) predicted the bond strength that is closer to the measured values, while that developed by Orangun *et al.* (1977, 1975) and Darwin *et al.* (1996) sometimes overestimated the bond strength between concrete and reinforcement.

As the analysis carried out in this study based on linear stress-strain behaviour, further research covering non-linear analysis, e.g., non-linear Finite Element analysis, would be a useful contribution to understand the mechanism involved in bond strength between reinforcement and concrete. Furthermore, due to the limited scope of the experimental programme, limited variations, e.g., size of beam and reinforcement, were tested. Therefore, it would be valuable carrying out testing programme that covers various sizes of beams with different reinforcement and various span.

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Notation

- A_{tr} : area of transverse reinforcement crossing plane of splitting adjacent to single anchored reinforcing bar
- c : minimum concrete cover or 1/2 the clear distance between the bars
- e : axial force eccentricity distance
- f'_c : concrete compressive strength (standard cylinder specimen)
- f_{ct} : tensile strength of concrete
- f_s : tensile stress in the reinforcing bar
- f_{su} : ultimate stress in reinforcing bar
- f_{yt} : yield strength of transverse reinforcement
- f_y : yield stress of reinforcing bar
- K : modulus of displacement
- K_{tr} : index of transverse reinforcement provided along anchored bar
- L_d : development length
- M : bond strength parameter given by Eq. (3)
- N : axial load
- P_{max} : maximum applied load
- R : K/f'_c , taken as 3 when ρ is close to 0.07
- s : spacing of transverse reinforcement
- T_c : concrete contribution to total force in bar at splice failure, $1b$
- u : average ultimate bond strength
- U : equivalent uniform bond stress at failure (bond strength)
- u_c : bond stress when the concrete cover cracks
- u_t : average bond stress corresponding to maximum applied load