Controlling the crack width of flexural RC members

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Reinforced concrete (RC) structures built using high strength deformed bars and designed using limit state design method were found to have larger crack widths. To control these crack widths and to enhance durability, different codes prescribe limiting crack widths based on the environment in which the structure exists. The latest revision of the Indian code stresses the importance of durability and has introduced formulae to calculate the crack widths. Unfortunately, the formulae given in the Indian code are complex and are seldom used in practice. A similar approach was used in the American code till 1999. However, recent research has found that there is no correlation between corrosion and crack widths. Also, there was a large scatter in the measured crack widths even in controlled laboratory experiments. Hence, a simple formula, involving the clear cover and calculated stress in reinforcement at service load has been included in the latest revision of the ACI code. A similar formula which also takes into account the effect of epoxy coating on reinforcement is suggested for the Indian code. Discussions on controlling the flexural cracking in the flanges of T-beams and side face reinforcement of large reinforced concrete beams are also included in the paper. The author highlights the need of introducing a simple formula for controlling crack widths in Indian codes on similar lines of the ACI code.

Keywords: *Crack width, flexural member, ACI code, durability, reinforcement spacing, side face reinforcement*

According to the design philosophy of the limit states method, two distinct classes of limit states should be satisfied, namely, ultimate limit states and serviceability limit states. While the former deals with safety in terms of strength, overturning, sliding, fatigue fracture, buckling, etc, the latter is concerned with serviceability in terms of deflection, cracking, durability, vibration, etc. The Indian code (clauses 42 and 43 of IS 456 : 2000) does not require the designer to perform any explicit check on deflection or crack width for all normal cases, provided the codal recommendations for limiting *l/d* ratios (for deflection control) and spacing of flexural reinforcement (for crack control) are complied with[1].

It has been recognised that many of the modern concrete structures are safe with respect to ultimate limit states. However, many times structural 'failures' are often reported in terms of serviceability. In particular, it is the serviceability limit state of durability that is often ignored all over the world. Inadequate durability is due to several factors such as improper production, placing, vibration and curing of concrete, chemical attack from the environment and resulting corrosion, inadequate sizes of structural members which result in excessive deflections and crack widths (and subsequent loss of durability). Hence it was given importance in the recent revision of the Indian code. An *Appendix* F was also added to calculate the crack width of flexural members. However, the equations gives in *Appendix* F of the code result in complex calculations. Moreover the calculated crack widths according to the formulae do not correlate with the crack width in members tested even under controlled laboratory experiments[2,3]. Hence the American code has dispensed with the crack width calculations and has given a much simpler equation to calculate the spacing of reinforcement – which will result in controlled cracking. Hence in this paper the drawbacks of the Indian code provisions are discussed and the need for introducing a formula similar to the ACI code is stressed. Other methods to control flexural cracking in the flanges of T-beams and deep flexural members are discussed.

Cracking in RC flexural members

Cracking in reinforced concrete members may be due to the following causes[4].

- Flexural tensile stress due to bending under applied loads

- Volume changes due to creep, shrinkage, thermal and chemical effects
- Additional curvatures due to continuity effects, settlement of supports, etc.

The discussion in this paper is confined to the cracking due to the first cause. It is difficult to predict the crack width due to the other two causes. However, it has been found that they are greatly controlled by good quality concrete, proper detailing of shrinkage and temperature reinforcement and proper location of expansion / control joints, etc.

It is of interest to note that structures built in the past using working stress design method and reinforcements with a yield strength of 250 MPa had low tensile stresses in the reinforcements at service loads. It has been found in laboratory investigations that cracking is generally proportional to the tensile stress in steel[5, 6]. Thus, with low tensile stresses in the reinforcements at service loads, these structures served their intended functions with very limited flexural cracking.

However, the introduction of high strength steel having yield stress of 415 MPa and higher, and the use of limit state design methods (which allow higher stresses in the reinforcements), result in visible cracks and hence the detailing of reinforcement to control cracking assumes more importance.

There are three perceived reasons for limiting the crack width in concrete. These are appearance, corrosion and water tightness. It may be particularly mentioned that all the three are not applicable simultaneously in a particular structure. Appearance is important in the case of exposed concrete for aesthetic reasons. Similarly, corrosion is important for concrete exposed to aggressive environments. Water tightness is required in the case of liquid storage structure and for marine/sanitary structures. Appearance requires limit of crack widths on the surface. This can be ensured by locating the reinforcement as close as possible to the surface (by using small covers) which will prevent cracks from widening. Corrosion control on the contrary requires increased thickness of concrete cover and better quality of concrete. Water tightness requires control on crack widths but applicable only to special structures. Hence, a single provision in the code is not sufficient to address the control of cracking due to all the above three reasons.

Table 1: Tolerable crack widths according to ACI 224R-80, CEB-FIP model code and IS code

Sl.No.	Exposure condition	Tolerable crack widths, mm		
		ACI 224 R-80	*CEB – FIP*	*IS 456 : 2000*
1	Low humidity, dry air or protective environment	0.40	0.4 – 0.6	0.30
2	High humidity, moist, air, soil	0.30	0.2– 0.3	0.20
3	Deicing chemicals	0.2	0.10 – 0.15	0.10
4	Sea water and sea water spray	0.15	0.10 – 0.15	0.10
5	Water-retaining structures	0.10	–	–

* Lower crack width limit is for cases with minimum cover; upper limit = 1.5 × minimum cover.

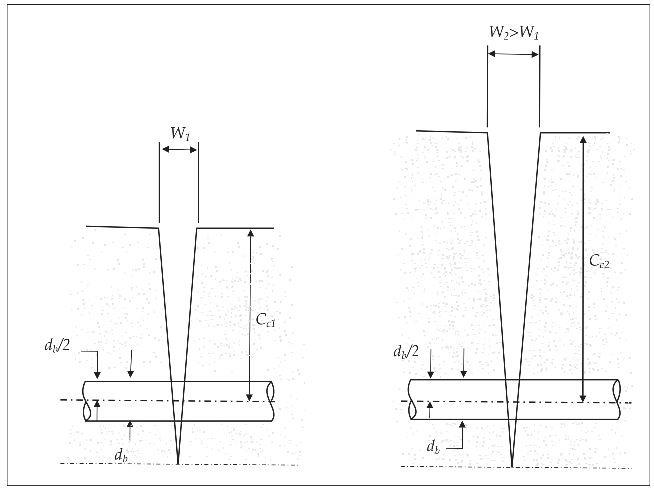

Fig 1 Crack width for difference cover thicknesses

Traditionally, the codes specified permissible crack widths in order to solve this problem. The permissible crack widths as specified by some codes are given in *Table* 1[1,4,7]. It is interesting to note that the earlier version of IS code recommended a limiting crack width of 0.004 times the nominal cover for severe environment.

Early investigations of crack width in beams and members subject to axial tension indicated that crack width was proportional to steel stress and bar diameter, but was inversely proportional to reinforcement percentage[3]. Other variables such as the quality of concrete, the thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar, depth of member and location of neutral axis, bond strength and tensile strength of concrete were also found to be important[2, 3]. Some of these factors are inter-related. There is a strong correlation between surface crack width and cover, c_c, as shown in *Fig* 1. For a particular magnitude of strain in the steel, the larger the cover, the larger will be the surface crack width affecting the appearance.

Indian code provisions

According to the explanatory handbook on Indian concrete code, the width of flexural crack at a particular point on the surface of a flexural member is found to increase with the increase in the following three major influencing factors[8]:

(*i*) average tensile strain at surface, which in turn, increases with increase in the mean tensile strain, ε_{sm}, in the neighbouring reinforcement;

(*ii*) distance between the point on the surface and the nearest longitudinal bar which runs perpendicular to the crack;

(*iii*) distance between the point on the surface and the neutral axis.

Due to the several inter-related variables, the estimate of the probable maximum width of surface cracks in a flexural member is a fairly complex problem. A number of widely different equations have been proposed (with semi-empirical

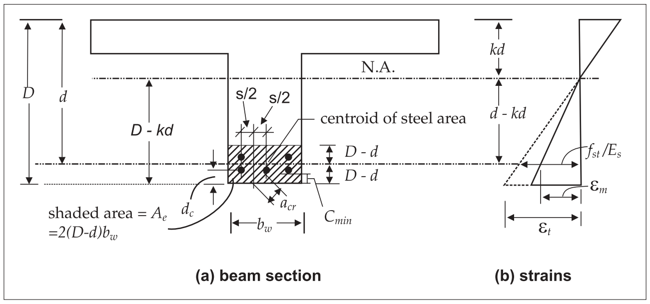

Fig 2 Parameters for crack width calculation

formulations) in the past[9]. The formulation given in *Appendix* F of the recent revision of the Indian code[1] is exactly similar to that given in the British code[10].

Provided the strain in tension reinforcement is limited to $0.8 f_y / E_s$, the design surface crack width is given by the code as:

$$w_{cr} = (3a_{cr}\varepsilon_m)/[1 + 2(a_{cr} - c_{\min})/(D - kd)] \quad \text{...(1)}$$

where,

a_{cr} = distance from the point considered to the surface of the nearest longitudinal bar (In *Fig* 2(a) $a_{cr} = [(0.5\,s)^2 + c^2_{min}]^{0.5}$ where s is the spacing between bars.

c_{min} = minimum cover to the longitudinal bar

ε_m = average steel strain at the level considered

D = overall depth of the member

kd = depth of neutral axis

E_s = the modulus of elasticity of the reinforcement, and

f_y = the yield stress of reinforcement.

Generally, the point for considering the maximum crack width is located on the surface of the beam or slabs (on the tension side), mid-way between two reinforcing bars as shown in *Fig* 2. The formula for mean strain, ε_m, is given by the code as

$$\varepsilon_m = \varepsilon_1 - \frac{b(D - kd)(a' - kd)}{3E_s A_{st}(d - kd)} \quad \text{...(2)}$$

where,

ε_1 = strain at the level considered, calculated ignoring the stiffening of the concrete in tension zone

b = width of the section at the centroid of the tension steel ($\mu = b_w$ for a rectangular web as in *Fig* 2)

a' = distance from the compression face to the point at which the crack width is being calculated

d = effective depth

A_{st} = area of tension steel.

A negative value for ε_m indicates that the section is uncracked.

American code formula

From 1971 through 1995, the American code formulation was based on formula suggested by Gergely and Lutz, which gives the maximum probable crackwidth as[5]:

$$w_{cr} = [(11\times10^{-6})\sqrt[3]{d_c(\frac{A_e}{n})}\beta]f_{st} \quad \text{... (3)}$$

where,

d_c = thickness of concrete cover measured from the extreme tension fibre to the centre of the nearest bar

A_e = effective area of concrete in tension surrounding the main tension reinforcement, having the same centroid as the tension steel

($A_e = 2\,(D\text{-}d)\,b_w$ in *Fig* 2)

n = number of bars in tension; in case different diameters are used, n shall be taken as the total steel area divided by the area of the largest bar diameter

f_{st} = stress at the centroid of the tension steel and may be taken as $0.6\,f_y$

β = ratio of the distances to the neutral axis from the extreme tension fibre and from the centroid of the reinforcement.

It has to be noted that equation 2 is empirical, and for dimensional homogeneity, the constant 3 in the denominator evidently has the inverse unit of stress. Similarly, the same applies to the constant 11 x 10^{-6} in equation 3, which was obtained from statistical analysis of the experimental data.

The 1995 edition of the code required that when the yield strength of reinforcement exceeds 275 MPa, detailing of the flexural tension reinforcement must be such that the following equation is satisfied

$$z = f_{st}\sqrt[3]{d_c\left(\frac{A_e}{n}\right)} \quad \text{...(4)}$$

≤ 30.6 kN/mm for interior exposures

≤ 25.4 kN/mm for exterior exposures

The above equation was derived from equation (3) with an approximate value of 1.2 for β and by dividing the crack width by the constant value, yielding the parameter, *z*. The numerical limitations of *z* for interior and exterior exposure, respectively, correspond to limiting crack widths of 0.4 mm and 0.33 mm. Ganesan and Sivananda compared the various formulae with the available experimental results and concluded that the best results are predicted by the formula proposed by Gergely and Lutz[9].

Discussion on crack width formulae

As already pointed out, increased cover results in increased crack widths at the surface. Similarly, increased cover is highly desirable from the point of view of durability and protection against corrosion of reinforcement. These two aspects appear to be contradictory. In order to comply with the ACI code specified z-factors, (equation 4), the method essentially encouraged reduction of the reinforcement cover, which could be detrimental to corrosion protection[3]. Moreover, the method severely penalised structures with covers more than 50 mm by either reducing the spacing or the service load stress of the reinforcement.

It has also been found that the role of cracks in the corrosion of reinforcement to be controversial. Research shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels[11,12]. Further, it has been found that actual crack widths in structures are highly variable[2,3]. A scatter of the order of ± 50 percent in crack widths was observed even in careful laboratory work[3]. Moreover, shrinkage and other time-dependent effects influence crack widths.

Exposure tests on concrete specimens indicated that concrete quality, adequate compaction and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface. Also, a better crack control was obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension. Hence, it was decided to replace the clause used to predict crack widths in field.

From the 1999 edition of the ACI code, the maximum bar spacing is specified directly. The spacing, s, of reinforcement closest to a surface in tension shall not exceed that given by

$$s = (95{,}000/f_{st}) - 2.5c_c \quad \text{... (5)}$$

but not greater than 300 ($252/f_{st}$)

where

s = centre to centre spacing of flexural tension reinforcement nearest to the extreme tension face, mm (where there is only one bar nearest to the extreme tension face, s is the width of the extreme tension face)

f_{st} = calculated stress in reinforcement at service load, computed as the unfactored moment divided by the product of the steel area and internal moment arm, MPa. It is permitted to take f_{st} as 60 percent of specified yield strength

c_c = clear cover from the nearest surface in tension to the surface of flexural tension reinforcement, mm.

Note that the spacing limitation is independent of the exposure condition and bar size used. Thus, for a required amount of flexural reinforcement, this approach would encourage use of smaller bar sizes to satisfy the spacing criteria of equation (5).

Table 2: Comparison of spacing of reinforcement, mm

Stress, MPa	As per equation no	Clear cover, mm 20	25	30	35	40	45	50	75
f_{st} = 250	5	300	300	300	292	280	267	255	192
	6	300	300	300	287	275	262	250	187
f_{st} = 275	5	273	275	270	258	245	233	220	158
	6	273	273	265	253	241	228	216	153

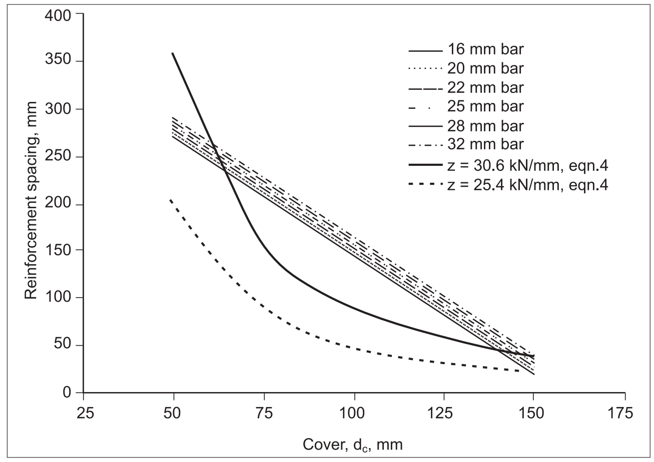

Fig 3 Comparison of maximum reinforcement spacing as per equations (4) and (5)

The maximum reinforcement spacing as per equations (4) and (5) for slabs with a single layer of reinforcement are compared in *Fig* 3[3]. It is seen that equation (5) significantly relaxes the spacing requirements for larger cover between 50 and 100 mm.

It has to be noted that equation (5) is not applicable to structures subject to very aggressive exposure or designed to be watertight. Special precautions are required and must be investigated for such cases.

From tests conducted by Treece and Jirsa it was found that epoxy coating significantly increased the width and spacing of cracks with the average width of cracks increasing up to twice the width of cracks in specimens with uncoated bar[13]. Hence the following equation was proposed for the maximum spacing of reinforcement, which included the effect of epoxy coated bars.

$$s = 300\,\alpha_s\left[1.25 - c_c/(120\,\alpha_s)\right] \le 300\,\alpha_s \quad \text{...(6)}$$

where

α_s = 250 γ_c/f_{st} = reinforcement factor

c_c = thickness of concrete cover

γ_c = reinforcement coating factor

= 1.0 for uncoated and 0.5 for epoxy coated bars

f_{st} = calculated stress in reinforcement at service level which may be taken as 0.60 f_y.

Table 3: Spacing and bar size for skin reinforcement

Depth, d, mm	Maximum spacing, S_{sk}, mm	Minimum bar area, A_b, at maximum spacing, mm^2
900	150	22.5
1050	175	52.5
1200	200	90.0
1500	250	187.50
1800	300	315.0
2100	300	405.0

The spacing calculated by this equations for uncoated bars is similar to the spacing calculated by using equation (5) (see *Table* 2) and has a format similar to that proposed by Frosch[14].

Distribution of tension reinforcement in flanges of I-beams

The American code also suggests that for control of flexural cracking in the flanges of T-beams, the flexural reinforcement must be distributed over a flange width not exceeding the effective flange width, or a width equal to 1/10 the span, whichever is smaller[2,3]. If the effective flange width is greater than 1/10 the span, additional longitudinal reinforcement as shown in *Fig* 4 should be provided in the outer portion of the flange.

Side-face reinforcement

The Indian code suggests that side face reinforcements shall be provided along the two faces, when the depth of the web in a beam exceeds 750 mm[1]. The total area of such reinforcement should not be less than 0.1 percent of the web area and should be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness, whichever is less[1].

ACI 318 building code stipulates special side face reinforcement in all beams that are deeper than 914 mm. Frantz and Breen proposed that the amount of side face reinforcement in large beams are independent of the amount of flexural reinforcement[16] and depend mainly on the member depth and also on the clear concrete cover to the side face reinforcement *C* and the diameter of the side face reinforcing bars, d_b. The current version of the American code suggests that the required skin reinforcement, *Fig* 5, must be uniformly distributed along both side faces of the member for a distance $d/2$ nearest the flexural tension reinforcement[2]. The spacing is not to exceed at least of $d/6$, 300 mm and 1000 $A_b/(d-750)$ where A_b is the area of an individual bar.

The total area of skin reinforcement provided on both faces need not be greater than one half the total area of the main tensile reinforcement. *Table* 3 gives maximum spacing and minimum bar area of that spacing.

It is of interest to note that the ACI code does not give any area for the skin reinforcement but only specified maximum spacing.

Adebar and Leeuwen compared different North-American requirements for side face reinforcement and shown that there are considerable differences regarding how much side-face reinforcement in appropriate[16]. They also concluded that

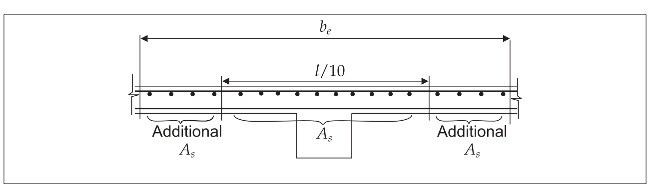

Fig 4 Negative moment reinforcement for flanged floor beams

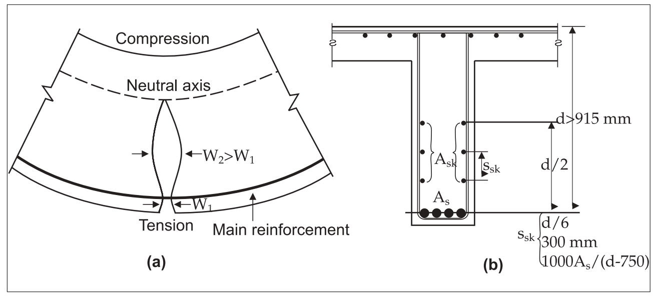

Fig 5 *(a)* Side face cracking and *(b)* Crack control "skin" reinforcement for deep beams

ACI code provision may not be conservative under certain exposure conditions when there are diagonal cracks. They also proposed an equation to predict the spacing of side face reinforcement.

Summary and conclusions

The deterioration of modern concrete structures resulted in the inclusion of durability concepts in the recent revision of the Indian code. Though several factors affect the durability, it was thought that by controlling the crack widths, the durability can be enhanced. In the recent revision an appendix was added to calculate the crack width of flexural members. However, due to the complexity of the equations, the design engineers seldom do these calculations. The American code also followed a similar formulation from 1971 through 1995. However recent research has established the fact that corrosion is not clearly correlated with surface crack widths in the stress ranges normally found with reinforcement at service load levels. Also a large scatter in the crack widths was found even in controlled laboratory tests. Hence a simple formula has been suggested in the 1999 edition of the American Concrete Institute. A similar formula, which also takes into account the effect of epoxy-coated reinforcement is suggested for the Indian code. Discussions on controlling the flexural cracking in the flanges of T-beams and side–face reinforcement detailing are also included.

The author strongly recommends that the crack width control formulae in the Indian codes may be reviewed on similar lines of ACI code.

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