Macro-Cracking and Crack Control in Concrete Structures —A State of the Art—

by E.G. Nawy

Synopsis: This paper presents the state of the art in evaluating flexural crack development and control of macro-cracking. It is based on extensive research over the past five decades in the United States and overseas in the area of macro-cracking in reinforced and prestressed concrete beams and two-way action slabs and plates.

With the advent of limit states theories that generally lead to economic proportioning of members, control of cracking has become essential in order to maintain the integrity and aesthetics of concrete structures. The trend is stronger than ever in better utilization of current concrete strengths, use of higher strength concretes, including super-strength concretes of 20,000 psi (138 MPa) compressive strength and higher, and increased application of prestressed concrete concepts. All these trends require closer control of serviceability requirements in cracking and deflection.

Design expressions are given for the control of cracking in reinforced concrete beams and thick one-way slabs, prestressed pretensioned and post-tensioned flanged beams and reinforced concrete two-way action structural floor slabs and plates. In addition, recommendations are given for the maximum tolerable flexural crack widths in concrete elements.

Keywords: Beams (supports); concretes; cover; cracking (fracturing); crack propagation; crack width and spacing; flexural strength; fracture properties; macrocracking; microcracking; post tensioning; prestressing; pretensioning; reinforced concrete; shrinkage; stabilization; volume change

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INTRODUCTION

Presently, the trend is stronger than ever in better utilization of concrete strength by use of super strength concretes of 20,000 psi (138 MPa) compressive strength and higher, use of high strength reinforcement, more prestressed concretes and increased use of limit states theories. These trends impose closer control of serviceability requirements which include cracking and deflection. Hence, knowledge of the mechanism of cracking in concrete elements becomes essential.

Concrete cracks early in its loading history. Most cracks are a result of the following actions to which concrete can be subjected:

1. Volumetric change caused by Plastic and drying shrinkage, creep under sustained load, thermal stresses and chemical incompatibility of concrete ingredients.

2. Stress due to bending, shear and other moments caused by transverse loads.

3. Direct stress due to applied loads or reactions or internal stress due to continuity, reversible fatigue load, long-term deflection, camber in prestressed systems, environmental effects, or differential movement in structural systems.

While the net result of these three actions is the formation of cracks, the mechanisms of their development cannot be considered identical. Volumetric change generates internal microcracking, which may develop into full cracking. Whereas, internal stresses due to applied loads could either generate internal microcracking, such as in fatigue due to load reversal, or flexural macrocracking leading to fully developed cracking.

This paper concentrates on the macro-cracking aspect of cracking behavior. Yet it is important to briefly discuss micro-cracking.

MICRO-CRACKING

Micro-cracking can be classified into two main categories: a) bond cracks at the aggregate-mortar interface, b) paste cracks within the mortar matrix. Interfacial bond cracks are caused by interface shear and tensile stresses due to early volumetric change without the presence of external load. Volume change caused by hydration and shrinkage could create tensile and bond stresses of sufficient magnitude as to cause failure at the aggregate-mortar interface (1). As external loads are applied, mortar cracks develop due to an increase in compressive stresses, propagating continuously through the cement matrix up to failure. Figure 1 is a typical schematic diagram that shows the nonlinear stress-strain relationship developed early in the stress history, starting with bond micro-cracking. While extensive work exists in the area of cracking due to volumetric change, the need is apparent for additional research on creep effects on micro-cracking and for the development of a universally acceptable fracture theory to interrelate the nonlinear behavioral factors resulting in crack propagation.

The damage to cement paste seems to play a significant role in controlling the stress-strain relationship in concrete. The coarse aggregate particles act as stress-raisers that decrease the strength of the cement paste. As a result, micro-cracks develop that can only be detected by large magnification. The importance of additional work lies not only in the evaluation of the micro-cracks, but also in their significance on the development of macro-cracks which generate from those micro-cracked centers of plasticity.

FLEXURAL CRACKING

External load results in direct and bending stresses causing flexural, bond and diagonal tension cracks. As the tensile stress in the concrete exceeds its tensile strength, internal micro-cracks form. These cracks develop into macro-cracks propagating to the external fiber zones of the element.

Upon the full development of the first crack in a reinforced concrete element, the stress in the concrete at the cracking zone is reduced to zero and the tension is assumed by the reinforcement (2). The distribution of ultimate bond stresses, longitudinal tensile stresses in the concrete and longitudinal tensile stress in the steel is schematically represented in Fig. 2.

Crack width is a primary function of the deformation of reinforcement between the two adjacent cracks, 1 and 2 in Fig. 2. The corresponding small concrete strain along the crack interval a_c may be neglected. The crack width would hence be a function of the crack spacing up to the load level at which no more cracks develop, leading to the stabilization of the crack spacing (Fig. 3).

The major parameters affecting the development and characteristics of the cracks are: percentage of reinforcement, bond characteristics, size of bar, concrete cover, and the concrete stretched area, namely, the concrete area in tension. On this basis, one can propose the following mathematical model for maximum crack width:

$$w = \alpha \, a_c^{\beta'} \, \epsilon_s^{\gamma} \qquad (1)$$

where α, β' and γ are nonlinearity constants. Crack spacing, a_c, is a function of the factors enumerated previously; it is inversely proportional to bond strength and active steel ratio in terms of the concrete area in tension. ϵ_s is the strain in the reinforcement induced by external load.

The basic mathematical modal in equation (1), with the appropriate experimental values of the constants α, β' and γ, can be derived for the particular type of structural member. Such a member can be a one-dimensional element such as a beam, a two-dimensional structure such as a two-way slab, or a three-dimensional member such as a shell or circular tank wall. Hence, it is expected that different expressions apply for the evaluation of the macro-cracking behavior of different structural elements consistent with their fundamental structural behavior (Ref. 1-10).

FLEXURAL CRACKING AND CRACK CONTROL IN REINFORCED CONCRETE BEAMS AND THICK ONE-WAY SLABS

Requirements for crack control in beams and thick one-way slabs (10 in. or thicker) in the ACI Building Code (ACI 318) are based on the statistical analysis of maximum crack width data from a number of sources. Based on that analysis, the following general conclusions were reached:

1. The steel stress is the most important variable.

2. The thickness of the concrete cover, although is an important variable, it is not the only geometric consideration..

3. The area of concrete surrounding each reinforcing bar is an important geometric variable.

4. The bar diameter is not a major variable.

5. The size of the surface crack width is influenced by the amount of strain gradient from the level of the steel to the tension face of the beam.

The simplified expression relating crack width to steel stress is given in (Ref. 4).

$$w_{max} = 0.076\ \beta\ f_s \sqrt[3]{d_c A} \times 10^{-3} \qquad (2)$$

where f_s = reinforcing steel stress, ksi

A = area of concrete symmetric with reinforcing steel divided by number of bars, in.^{2}

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire closest thereto, in.

β = h_2/h_1, where, h_1 = distance from the neutral axis to the centroid of all thereinforcing steel, in.

and h_2 = distance from neutral axis to extreme concrete tensile surface.

A plot relating the reinforcement strength to the ratio of the concrete area in tension to the reinforcement area is shown in Fig. 4 for all bar sizes.

In the ACI Code, when the design yield strength, f_y, for tension reinforcement exceeds 40 ksi, cross sections of maximum positive and negative moment have to be so proportioned that the quantity z given by Eq. 3.

$$z = f_s \sqrt[3]{d_c A} \tag{3}$$

does not exceed 175 kips per in. for interior exposure and 145 kips per in. for exterior exposure. Calculated stress in the reinforcement at service load f_s(ksi) shall be computed as the moment divided by the product of steel area and internal moment arm. In lieu of such computations, it is permitted to take f_s as 60 percent of specified yield strength f_y.

When the strain, in the steel reinforcement, ϵ_s, is used instead of the stress, f_s, Eq. (3) becomes

$$w = 2.2\, \beta\, \epsilon_s \sqrt[3]{d_c A} \tag{4}$$

Eq. (4) is valid in any units of measurement.

The cracking behavior in thick one-way slabs is similar to that in shallow beams. For one-way slabs having a clear concrete cover in excess of 1 in. (25.4 mm), Eq. 4 can be adequately applied if β = 1.25 to 1.35 is used.

<u>CEB Recomendations</u>

Crack control recommendations proposed in the European Model Code for Concrete Structures are supposed to apply to both prestressed and nonprestressed concrete members. The basic concept is summarized in the following paragraphs.

The mean crack width, w_m, in beams is expressed in terms of the mean crack spacing, s_{rm}, and the average strain in the steel, ϵ_{sm}, such that

$$w_m = \epsilon_{sm} s_{rm} \tag{5}$$

where
$$\epsilon_{sm} = \frac{f_s}{E_s}\left[1 - \chi \left(\frac{f_{sr}}{f_s}\right)^2\right] \leq 0.4 \frac{f_s}{E_s} \tag{6}$$

f_s = steel stress at the crack location

f_{sr} = steel stress at the crack location due to forces causing cracking at the modulus of rupture tensile strength of concrete

χ = bond coefficient, = 1.0 for ribbed bars, reflecting influence of load repetitions and load duration

The mean crack spacing is

$$s_{rm} = 2(c - \frac{s}{10}) + \chi_2 \chi_3 \frac{d_b}{Q_R} \qquad (7)$$

where

c = clear concrete cover

s = bar spacing, limited to $15d_b$

χ_2 = 0.4 for ribbed bars

χ_3 = depends on the shape of the stress diagram, 0.125 for bending

Q_R = A_s/A_t

A_t = effective area in tension, depending on arrangement of bars and type of external forces; it is limited by a line $c + 7d_b$ from the tension face for beams; in the case of thick slabs, not more than half the distance from the tension face to the neutral axis

A simplified formula can be derived for the mean crack width in beams with ribbed bars.

$$w_m = 0.7 \frac{f_s}{E_s} (3c + 0.05 \frac{d_b}{Q_R}) \qquad (8)$$

A characteristic value of the crack width, presumably equivalent to the probable maximum value, is given as $0.7w_m$.

The 1991 CEB model code changes on crack width evaluation have modified Eqs. 5 and 7 to Eqs. 5a and 7a as follows:

$$w_k = \ell_{s,max}(\epsilon_{sm} - \epsilon_{cm} - \epsilon_{cs}) \qquad (5a)$$

where

ϵ_{sm} = average steel strain over length $\ell_{s,max}$

ϵ_{cm} = average concrete strain over length $\ell_{x,max}$

ϵ_{cs} = strain of concrete due to shrinkage

and

$$\ell_{s,max} = 2(s_o + \frac{\sigma_{s2} - \sigma_{sE}}{4\tau_{bk}} \phi_s) \tag{7a}$$

s_o = length at crack vicinity along which bond stresses have negligiable values

σ_{s2} = steel stress in crack

σ_{sE} = steel stress at point of zero slip

τ = $2f'_t$ for deformed bars

b_k = $1\ f'_t$ for plain bars

ϕ_s = steel bar diameter

It should be noted in all these expressions for cracking in beams that they are all based on concrete compressive strength of 4000-5000 psi. The crack widths for higher strength concretes (in excess of 6000 psi - 41.4 MPa) are expected to decrease proportionately, although not linearly.

FLEXURAL CRACKING AND CRACK CONTROL IN PRESTRESSED PRETENSIONED AND POST-TENSIONED BEAMS

Partial prestressing is increasingly used whereby limited tensile stresses are allowed in the concrete under service load conditions. In such designs, practicality and economy cause use of nonprestressed steel to carry such tensile stresses. Consequently, an evaluation of the flexural crack widths and spacing and control of their development have become essential. Work in this area is relatively limited because of the various factors affecting crack width development in prestressed concrete. However, experimental investigations support the hypothesis that the major controlling parameter is the reinforcement stress change beyond the decompression stage. Nawy, et al (11,12), have undertaken extensive research since the 1960s on the cracking behavior of prestressed pretensioned and post-tensioned beams and slabs. Their investigations were prompted by the fact that corrosion in highly stressed prestressing steel results in premature significant loss of prestress. Serviceability behavior under service and overload conditions can be controlled through the application of the criteria presented in this section.

Mathematical Model Formulation for Serviceability Evaluation

Crack spacing -- Primary cracks form in the region of maximum bending moment when the external load reaches the cracking load. As loading is increased, additional cracks will form, until the stress in the concrete no longer exceeds its tensile strength at the proximity of the original cracks, whereupon the formation of new cracks will cease regardless of an increase in load intensity. This condition produces the absolute minimum crack spacing at the corresponding high steel stress level and is termed the "stabilized minimum crack spacing". The maximum possible crack spacing under this stabilized condition is twice the minimum, and is termed the "stabilized maximum crack spacing". Hence, the stabilized mean crack spacing a_{cs} is deduced as the mean value of the two extremes.

The total tensile force, T, transferred from the steel to the concrete over the stabilized mean crack spacing can be defined as

$$T = \gamma \, a_{cs} \, \mu \, \Sigma o \qquad (9a)$$

where

γ = a factor reflecting the distribution of bond stress

μ = maximum bond stress which is a function of $\sqrt{f'_c}$

Σo = sum of the circumference of the reinforcing elements including reinforcing bars and prestressing strands

The resistance, R, of the concrete area A_t in tension can be defined as

$$R = A_t \, f'_t \qquad (9b)$$

where f'_t = tensile splitting strength of the concrete. By equating Eqs. 9a and 9b, the following expression for a_{cs} is obtained,

$$a_{cs} = c \, \frac{A_t \, f'_t}{\Sigma o \, \sqrt{f'_c}} \qquad (10a)$$

where c is a constant to be developed from the tests. Fig. 5 shows the force distributions that result in the stabilized crack spacing.

The concrete stretched area, namely the concrete area in tension, A_t, for both the evenly distributed and non-evenly distributed reinforcing elements, is illustrated in Fig. 6. With a mean value of $f'_t / \sqrt{f'_c} = 7.95$ in this investigation, a regresson analysis of the test data resulted in the following expression for the mean stabilized crack spacing

$$a_{cs} = 1.20\ A_t/\Sigma o \qquad (10b)$$

Crack width -- At any load level where cracks occur, the net stress in the prestressed tendon or the magnitude of the tensile stress in the normal mild steel after decompression, Δf_s (ksi) is expressed as:

$$\Delta f_s = f_{nt} - f_d \qquad (11)$$

where

f_{nt} = stress in the prestressing steel at any load beyond the decompression load

f_d = stress in the prestressing steel corresponding to the decompression load. Here decompression is taken as the reference point, where the stress in the concrete at the level of the center of gravity of all the reinforcing steel, is zero.

The unit strain $\epsilon_s = \Delta f_s/E_s$. It is logical to disregard as insignificant the unit strains in the concrete due to the effects of temperature, shrinkage and elastic shortening. The maximum crack width as defined in Eq. 1 can be taken as

$$w_{max} = k\ a_{cs}\ \epsilon_s^{\alpha} \qquad (12a)$$

where k and α are constants to be established by tests, or Eq. 12a can be expressed in terms of Δf_s as follows

$$w_{max} = k'\ a_{cs}(\Delta f_s)^{\alpha} \qquad (12b)$$

where k' is a constant in terms of constant k.