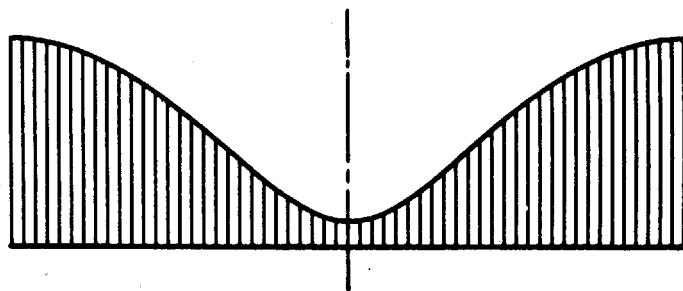
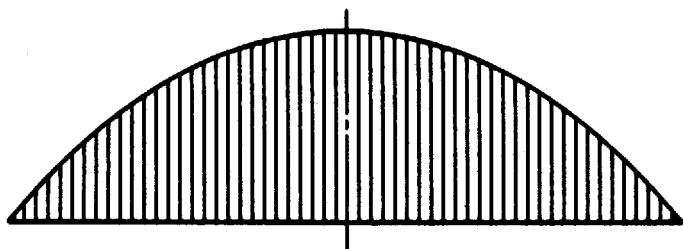
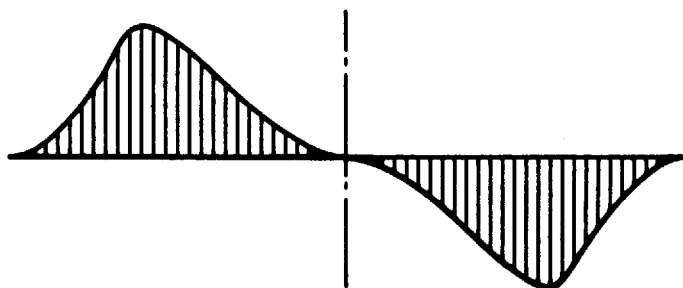


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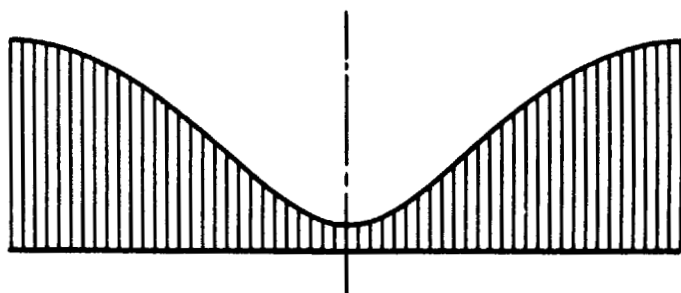
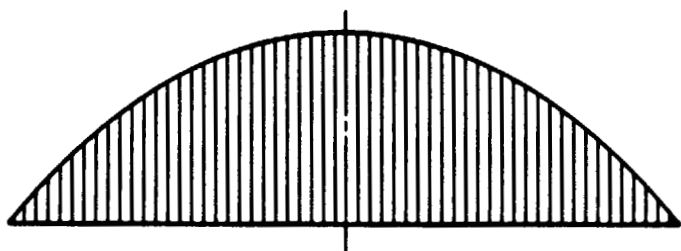
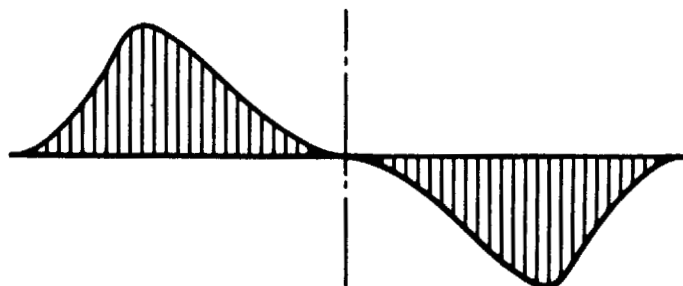


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DESIGNING CONCRETE STRUCTURES FOR SERVICEABILITY AND SAFETY



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DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is March 1, 1993. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the September/October 1993 issue of either ACI Structural Journal or ACI Materials Journal depending on the subject emphasis of the individual paper.

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PREFACE

Design for serviceability and safety is central to the work of structural engineers, code-writing bodies and the users. The current era of high strength materials, exotic additives and limit states of design has necessitated better control of constructed facilities in their short and long-term behavior at service load and at ultimate load. This Special Publication concentrates on topics that give the design engineer and contractor an insight into how to avoid practices that could affect the integrity or long-term performance of structural elements and systems.

The text is the outgrowth of a national symposium of the American Concrete Institute co-sponsored by ACI Committees 348 and 435, and covers topics ranging from crack-control in reinforced and prestressed concrete, safety provisions in design codes and practical deflection computations to limit state design principles and seismic performance of frame structures. Several papers that could not be presented due to time limitations are included.

The papers dealing with serviceability, highlight requirements of the ACI Codes and Reports in addition to relevant state of the art developments. The papers covering safety deal with issues ranging from philosophical discussions of treatment of safety in codes to project case studies. Overlap is expected since serviceability and safety are indivisible.

All the papers presented in this publication were reviewed by recognized experts in accordance with the ACI review procedures. It is hoped that designers, constructors and codifying bodies will be able to draw on the material presented in improving the safety and long-term cracking and deflection behavior of concrete constructed facilities.

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SP 133-1

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:

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

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).

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

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.

$\beta = h_2/h_1$, 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.

$$z = f_s \sqrt[3]{d_c A} \quad (3)$$

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

$$w = 2.2 \beta \epsilon_s \sqrt[3]{d_c A} \quad (4)$$

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 $\beta = 1.25$ to 1.35 is used.

CEB Recommendations

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

$$w_m = \epsilon_{sm} s_{rm} \quad (5)$$

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

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

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

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.

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

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:

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

where

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

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

ϵ_{cs} = strain of concrete due to shrinkage

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

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

σ_{s2} = steel stress in crack

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

$\tau = 2f'_t$ 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

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

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

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

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

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

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 regression analysis of the test data resulted in the following expression for the mean stabilized crack spacing

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

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:

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

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

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

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

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

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

Expressions for Pretensioned Beams

Analysis of the test data of the simply supported test beams in this work thus leads to the following expression of crack width at the center of gravity of the reinforcement:

$$w_{\max} = 1.4 \times 10^{-5} a_{cs} (\Delta f_s)^{1.31} \quad (13)$$

Introducing Eq. 10b and linearizing Eq. 13 for easier use by the design engineer leads to the following simplified expression of the maximum crack width at the center of gravity of the reinforcing steel:

$$w_{\max} = 5.85 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (14a)$$

The maximum crack width at the tensile face of the concrete section may be expressed as:

$$w'_{\max} = 5.85 \times 10^{-5} R_i \frac{A_t}{\Sigma o} (\Delta f_s) \quad (14b)$$

where R_i is the distance ratio as defined in the notations.

It is to be noted that the crack width increases linearly from the steel level to the concrete tension face. Figure 7 gives typical results relating the ratio between these two levels. This ratio is very close to the distance ratio R_i in Eqs. 14b and 15b.

A plot of the test data is given in Fig. 8. Eq. 14a is derived from the test data as a best fit expression. The spread of 40 percent shown in the figure is reasonable in view of the randomness of crack development and the linearization effects of the original expression of Eq. 13.

Expressions for Post-Tensioned Beams

A similar expression is developed for the crack width at the center of gravity of the total reinforcement in post-tensioned bonded beams containing mild steel bars as follows:

$$w_{\max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (15a)$$

for the width at the reinforcement level closest to the tensile face, and the maximum crack width at the tensile face of the concrete section may be expressed as:

$$w_{\max} = 6.51 \times 10^{-5} R_i \frac{A_t}{\Sigma o} (\Delta f_s) \quad (15b)$$

For nonbonded beams, the factor 6.51 in Eqs. 15a and 15b becomes 6.83.

A plot of the data and the best fit expression for Eq. 15a is given in Fig. 9.

A typical plot of the effect of the various steel percentages on the crack spacing at the various stress levels Δf_s is given in Fig. 10. It is seen from this plot that crack spacing stabilizes at a net stress level of 30 to 34 ksi.

Cracking of Higher Strength Prestressed Beams

Recent work was completed at Rutgers University (1991) on the cracking behavior of prestensioned and nonbonded post-tensioned beams having cylinder compressive strengths in the range of 10,200 psi to 12,300 psi (70.3 to 84.8 MPa). Analysis of the test data leads to introducing modifying multipliers of a value less than 1.0 in order to include this effect of the high concrete strength on the stabilized crack width in prestressed beams.

A multiplier λ_r should be used in Eqs. 14a and 14b for pretensioned beams and λ_o should be used in Eqs. 15a and 15b for nonbonded post-tensioned beams.

$$\lambda_r = \frac{2}{(0.75 + 0.06 \sqrt{f'_c}) \sqrt{f'_c}} \quad (16a)$$

and

$$\lambda_o = \frac{1}{0.75 + 0.06 \sqrt{f'_c}} \quad (16b)$$

Other Work on Cracking in Prestressed Concrete

Naaman (8) arrived at the following modified expression for crack width in pretensioned members by regression analysis of the test results of various investigators.

$$w_{\max} = [42 + 5.58 \frac{A_t}{\Sigma o} (\Delta f_s)] \times 10^{-5} \quad (17)$$

This expression is similar to Eq. 14 by the author. Fig. 11 is a plot of Eq. 17 against the experimental results of the various researchers; it yields an excellent fit.

A comparison is made between the author's equations and those of CEB-FIP using similar notations as follows:

$$\text{Nawy:} \quad w_{\max} = \xi \left(\frac{A_t}{\Sigma o} \right) \Delta \sigma_p$$

$$\text{CEB-FIP:} \quad w_{\max} = \left[K \frac{\phi}{\rho_t} + \alpha_o \right] \frac{\Delta \sigma_p}{E_s} \left[1 - \beta \left(\frac{\Delta \sigma_{pr}}{\Delta \sigma_p} \right)^2 \right]$$

where

$1/\Sigma o = \lambda \left(\frac{\phi}{\rho_t} \right)$ using λ as a multiplier in lieu of K and β as test parameters, ϕ being the reinforcement diameter.

The terms α_o and $\left(\frac{\Delta \sigma_{pr}}{\Delta \sigma_p} \right)^2$ in the CEB-FIP expression are not of major significance and are accounted for in the constant $\xi = 6.51$ for the post-tensioned beams in the author's expression.

The expressions reported in the study (10) concentrating on the area of concrete in tension and the nominal strain in the concrete at the tensile face would not give a reliable prediction of the crack width. In particular it does not account for the actual stress in the steel reinforcement but depends on prediction of strain at the concrete surface, which is difficult to reliably evaluate.

CRACKING AND CRACK CONTROL IN TWO-WAY ACTION SLABS AND PLATES

The control of flexural cracking is essential in structural floors, many of which are under two-way action, such as in office and industrial buildings, schools, and parking garages, where cracks at service load and overload conditions can be serious. In such structures, the design service loads and overloads exceed those in normal size apartment buildings and they are often exposed to more adverse environmental conditions.

Flexural Cracking Mechanism and Fracture Hypothesis

Flexural cracking behavior in concrete structural floors under two-way action is significantly different from that in floors under one-way action. Crack-control equations derived for beams underestimate crack widths when applied to two-way slabs and plates, and they do not provide the designer with guidelines on spacing. Cracking in two-way slabs and plates is affected primarily by the steel stress level and the spacing of the reinforcement in the two perpendicular directions. In addition, the clear concrete cover in two-way slabs and plates is nearly constant at 0.75 in. (19 mm) for interior exposure, whereas it is a major variable in the crack-control equations for beams. The

results of extensive tests on slabs and plates by Nawy et al, demonstrate this difference in behavior in a fracture hypothesis on crack development and propagation in two-way plate action. As seen in Fig. 12, stress concentration develops initially at the points of intersection of the reinforcing bars and at the welded joints of the wire mesh, that is, at the grid nodal points. Thus, fracture lines are dynamically generated along the paths of least resistance, namely, along A_1B_1 , A_1A_2 , A_2B_2 , and B_2B_1 . When the nodal points A_1 , B_1 , A_2 , and B_2 are spaced close enough a repetitive initial fracture grid of narrow orthogonal cracks is formed. If the spacing of the reinforcing grid intersections is too large, the magnitude of stress concentration and the energy absorbed per unit grid becomes too low to generate cracks along the reinforcing wires or bars. As a result, the principal cracks follow diagonal yield-line cracking in the plain concrete field, away from the reinforcing bars, early in the loading history. These cracks are wide and few.

This hypothesis also leads to the conclusion that surface deformations of the individual reinforcing elements have little effect in arresting the generation of the cracks or controlling their type or width in a two-way-action slab or plate. Similarly, since the cracking grid would be a reflection of the reinforcement grid at the preferred orthogonal narrow cracking pattern it may be concluded that the scale effect on two-way-action cracking behavior is insignificant. Therefore, to control cracking in two-way-action floors, the major parameter to be considered is the reinforcement spacing in the two perpendicular directions. Concrete cover has only a minor effect, since it is usually a constant value, usually, 0.75 in. (20 mm).

For a given area of steel determined for bending in one direction, that is, for energy absorption per unit slab area, the smaller the spacing of the transverse bars or wires, the smaller should be the diameter of the longitudinal bars. The reason is that less energy has to be absorbed by the individual longitudinal bars. If one considers that the magnitude of fracture is determined by the energy imposed per specific volume of reinforcement acting on a finite element of the slab, a proper choice of the reinforcement grid size and bar size can control cracking and render it into a preferred orthogonal grids system.

It must be emphasized that this hypothesis is important for serviceability and reasonable overload conditions. In relating orthogonal cracks to yield-line cracks, the failure of a slab ultimately follows the generally accepted rigid-plastic yield-line criteria.

Crack Control Equation

The basic expression for relating crack width to strain in the reinforcement is given by Eq. 1 and repeated as follows:

$$w = \alpha a_c^{\beta'} \epsilon_s^\gamma \quad (18)$$

The effect of the tensile strain in the concrete between the cracks is neglected as insignificant, a_c is the crack spacing, ϵ_s the unit strain in the reinforcement, and α , β' , and γ are constants. As a result of the fracture hypothesis presented, the mathematical model in Eq. 18 and the statistical analysis of the data of 90 slabs tested to failure, the following crack-control equation is derived:

$$w = K \beta f_s \sqrt{G_I} \quad (19)$$

where the grid index $G_I = \frac{d_{b1} s_2}{\rho_{t1}}$ is a measure of the reinforcement distribution in the two-way action slab or plate. The grid index G_I can be transferred in Eq. 19 to

$$G_I = \frac{s_1 s_2^d c}{d_{b1}} \frac{8}{\pi} \quad (20)$$

where K = fracture coefficient, having a value of $K = 2.8 \times 10^{-5}$ for uniformly loaded restrained two-way-action square slabs and plates. For concentrated loads or reactions, or when the ratio of short to long span is less than 0.75 but larger than 0.5, a value of $K = 2.1 \times 10^{-5}$ is applicable. For a span aspect ratio of 0.5, $K = 1.6 \times 10^{-5}$. Units of coefficient K are in square inch per lb.

β = ratio of the distance from the neutral axis to the tensile face of the slab to the distance from the neutral axis to the centroid of the reinforcement grid (to simplify the calculations use $\beta = 1.25$, although it varies between 1.20 and 1.35)

f_s = actual average service load reinforcement stress level, or 40% of the design yield strength, ksi

d_{b1} = diameter of the reinforcement in direction 1 closest to the concrete outer fibers, in.

s_1 = spacing of the reinforcement in direction 1, in.

s_2 = spacing of the reinforcement in perpendicular direction 2, in.

1 = direction of the reinforcement closest to the outer concrete fibers; this is the direction for which crack control check is to be made

ρ_{t1} = active steel ratio in direction 1

area of steel A_s per foot width

$$= \frac{12(d_{b1} + 2c_1)}{12(d_{b1} + 2c_1)}$$

where c_1 is clear concrete cover measured from the tensile face of the concrete to the nearest edge of the reinforcing bar in direction 1

w = crack width at face of concrete caused by flexural load, in.

Subscripts 1 and 2 pertain to the directions of reinforcement. Detailed values of the fracture coefficients for various boundary conditions are given in Table 1.

A graphical solution of Eq. 19 is given in Fig. 13 for rapid determination of the reinforcement size and spacing needed for crack control using,

$$f_y = 60 \text{ ksi (414 MPa) and}$$

$$f_s = 40\% f_y = 24 \text{ ksi (165.5 MPa)}$$

Although crack width in two-way action is mostly controlled by the spacing of the reinforcement grid intersections and not by concrete strength, the expression for crack width could become slightly conservative if the slab or plate concrete cylinder compressive strength exceeds 6000 psi. For economical utilization of slab concrete strength, it is highly unlikely that two-way slab floors in normal structures would be justifiably built having concrete strength much higher than 6000 psi.

Tolerable Crack Widths in Concrete Structures

The maximum crack width that a structural element should be permitted to develop depends on the particular function of the element and the environmental conditions to which the structure is liable to be subjected. Table 2 from the ACI Committee 224 report on cracking serves as a reasonable guide on the tolerable crack widths in concrete structures under the various environmental conditions that are normally encountered.

The crack control equation and guidelines presented are important not only for the control of corrosion in the

reinforcement but also for deflection control. The reduction of the stiffness, EI , of the two-way slab or plate due to orthogonal cracking when the limits of permissible crack widths in Table 2 are exceeded, can lead to excessive deflection both short-term and long-term. Deflection values several times those anticipated in the design, including deflection due to construction loading, can be reasonably controlled through camber and control of the flexural crack width in the slab or plate. Proper selection of the reinforcement spacing s_1 and s_2 in the perpendicular directions as discussed in this section, not to exceed 12 in. (30 cm) center to center, can maintain good serviceability performance of a slab system under normal and reasonable overload conditions.

Long-term effects on cracking -- In most cases, the magnitude of crack widths increases in long-term exposure and long-term loading. The increase in crack width can vary considerably under cyclic loading, such as in bridge decks. However, crack width increases at a decreasing rate with time. In most cases, a doubling of crack width after several years under sustained loading can be expected.

CONCLUSIONS

With the aid of the expressions presented above and summarized herein, the design engineer and the constructor can limit the flexural macro crack width that develops in concrete systems. By limiting the width to within the permissible levels given in Table 2 in accordance with the prevailing environmental conditions, it would be possible to prevent or considerably minimize long-term corrosion deterioration and also maintain the aesthetic integrity of the various elements of the system.

Reinforced Concrete Beams and Thick One-Way Slabs

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

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

and f_s is in ksi. The value of z is not to exceed 145 kip per inch for exterior exposure and 175 kip per inch for interior exposure.

Prestressed Pretensioned Beams

a) At the center of gravity of steel reinforcement level

$$w_{\max} = 5.85 \times 10^{-5} \frac{A_t}{\sum_0} (\Delta f_s)$$

b) At the tensile face of the concrete

$$w'_{\max} = 5.85 \times 10^{-5} R_1 \frac{A_t}{\Sigma_o} (\Delta f_s)$$

Prestressed Post-Tensioned Beams

a) At the center of gravity of the steel reinforcement level
for bonded beams

$$w_{\max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma_o} (\Delta f_s)$$

b) At the tensile face of the concrete element

$$w_{\max} = 6.51 \times 10^{-5} R_1 \frac{A_t}{\Sigma_o} (\Delta f_s)$$

For nonbonded beams, the factor 6.51 becomes 6.83.

Use the modifying multipliers of Eqs. 16a and 16b for beams of higher strength concrete.

These expressions are applicable to both fully and partially prestressed beams.

Two-Way Action Structural Slabs and Plates

$$w_{\max} = K \beta f_s \sqrt{G_1}$$

where

$$G_1 = \frac{s_1 s_2 d_c}{d_{b1}} \times \frac{8}{\pi}$$

The values of coefficient K are as follows

K	Fully restrained slabs and plates
2.8×10^{-5}	Uniformly loaded, square
2.1×10^{-5}	At concentrated loads and columns
2.1×10^{-5}	$0.5 > \ell_s/\ell_\ell < 0.75$
0.6×10^{-5}	$\ell_s/\ell_\ell < 0.5$

*For simply supported slabs multiply above values by 1.6. interpolate K values for intermediate span ratios between 1.0 and $0.75 \ell_s/\ell_\ell$ or for partial restraints at the boundaries such as cases of end and corner panels of multipanel floor systems. ℓ_s and ℓ_ℓ are respectively the short and long spans of the two-way slab or plate.

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METRIC (SI) UNIT EQUIVALENTS

1 inch	= 25.4 mm
1 foot	= 0.305 m
1 in. ²	= 645.16 mm ²
1 in. ³	= 16,387.06 mm ³
1 in. ⁴	= 416.231 mm ⁴
1 psi	= 6.895 Pa
1 ksi	= 6.895 MPa
1 lb	= 4.448 N
1 kip	= 4448 N
1 lb/ft	= 14.594 N/m
1 kip/ft	= 14.594 kN/m
1 kip-in	= 113 N-m
1 $\sqrt{f'_c}$ psi	= 0.083036 $\sqrt{f'_c}$ MPa

NOTATION

A_g = gross area of concrete, in.²

A_s = area of tension reinforcement, in.²

- A_t = the effective concrete area in uniform tension, in.²
 In the case of two-way slabs and plates, it is =
 $12 (d_{b1} + 2c_1)$ for a 12 in. strip, defining the
 concrete area stretched in tension, in.²
- a_{cs} = stabilized mean crack spacing, in.
- c = clear concrete cover, in.
- CEB = Comité Euro-International du Béton
- d_b = diameter of reinforcing bar or wire, in.
- d_c = effective depth to center of the first layer of reinforcement
- E_c = modulus of elasticity of concrete, psi, ksi
- E_s = modulus of elasticity of steel, psi, ksi
- f'_c = cylinder compressive strength of concrete, psi
- f_y = yield strength of the reinforcement by the 0.2% offset, ksi
- f_s = service load level steel stress = $0.6 f_y$ in beams and
 $0.4 f_y$ in two-way action slabs and plates, ksi
- f'_t = tensile splitting strength of concrete (ASTM C 496-62T), psi
- F = prestressing force, lb., kip
- f_d = stress in the prestressing steel corresponding to the decompression load, ksi
- f_{nt} = stress in the prestressing steel at any load level beyond the decompression load, ksi
- f_r = modulus of rupture of concretes, psi

- G_{I1} = grid index in direction "1" ($= d_{b1}s_2/\rho_{t1}$) closest to outer, in.²
- G_{I2} = grid index in direction "2" ($= d_{b2}s_1/\rho_{t2}$) in direction perpendicular to "1", in.²
- h_1 = distance from the neutral axis to the centroid of the reinforcement, in.
- h_2 = distance from the neutral axis to the tension face, in.
- h = total slab of plate thickness, in.
- K = fracture coefficient dependent on load or reaction type and on the support condition
- k = kip, 1000 lb.
- ksi = kips per sq. inch
- Δf_s = net stress in the prestressing steel or the magnitude of tensile stress in the mild steel after decompression, ksi
- λ_r, λ_o = modifying multipliers for crack width in high strength prestressed beams
- ρ = steel percentage A_s/bd in beams
- ρ_{t1} = active steel ratio
 area of steel A_s per foot width
 $= \frac{12(d_{b1} + 2c_1)}{12(d_{b1} + 2c_1)}$
- R_i, β = distance ratio h_2/h_1
- s = spacing of reinforcement, in.
- $w_{\max}$ = maximum crack width in inches at concrete face, in.
- "1" = direction of reinforcing elements closest to outer concrete tensile fibers, for which crack control check is to be made

- α, β', γ = coefficients of regression function in slabs
- Σo = sum of perimeters of all effective bars and tendons crossing the section on the tension side, in.
- T = total tensile force transferred by bond from steel, lb., kip
- μ = bond strength in concrete, psi
- w = maximum crack width at steel level, in.
- y_t = distance from the centroidal axis of gross section to the extreme fiber in tension, in.
- Z = quantity limiting distribution of flexural reinforcement in beams and thick one-way slabs = $f_s \sqrt[3]{\frac{A}{c}}$, kip/inch

TABLE 1 -- FRACTURE COEFFICIENTS FOR SLABS AND PLATES

Loading type ^a	Slab shape	Boundary condition ^b	Span ratio, ^c S/L	Fracture coefficient 10 ⁻⁵ K
A	Square	4 edges r	1.0	2.1
A	Square	4 edges s	1.0	2.1
B	Rectangular	4 edges r	0.5	1.6
B	Rectangular	4 edges r	0.7	2.2
B	Rectangular	3 edges r, 1 edge h	0.7	2.3
B	Rectangular	2 edges r, 2 edges r	0.7	2.7
B	Square	4 edges r	1.0	2.8
B	Square	3 edges r, 1 edge h	1.0	2.9
B	Square	2 edges r, 2 edges h	1.0	4.2

^aLoading type: A, concentrated; B, uniformly distributed.

^bBoundary condition: r, restrained; s, simply supported; h, hinged.

^cSpan ratio S/L: S, clear short span; L, clear long span.

TABLE 2 -- MAXIMUM TOLERABLE FLEXURAL CRACK WIDTHS

Exposure condition	Crack width	
	in.	mm.
Dry air or protective membrane	0.016	0.40
Humidity, moist air, soil	0.012	0.30
De-icing chemicals	0.007	0.18
Seawater and seawater spray; wetting and drying	0.006	0.15
Water-retaining structures (excluding nonpressure pipes)	0.004	0.10

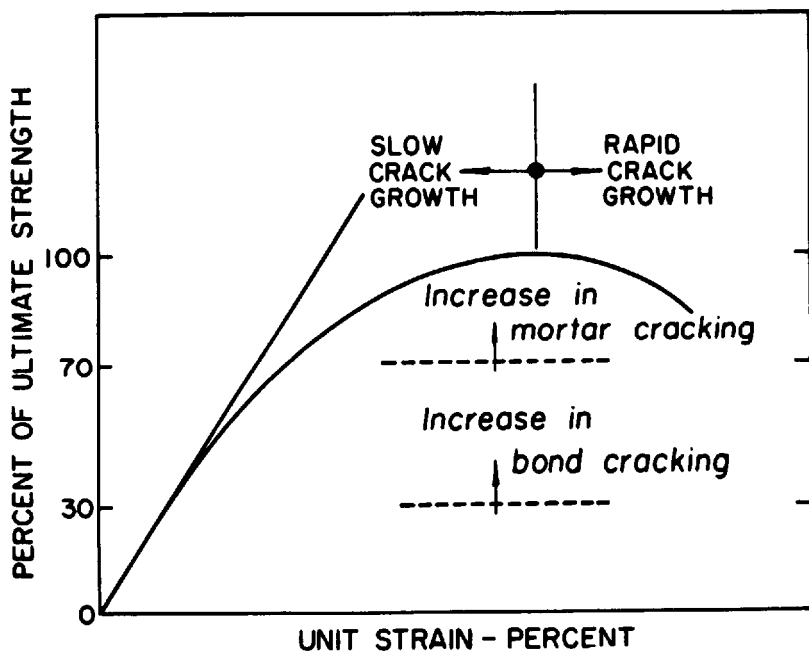


Fig. 1--Schematic stress strain diagram of concrete in microcracking

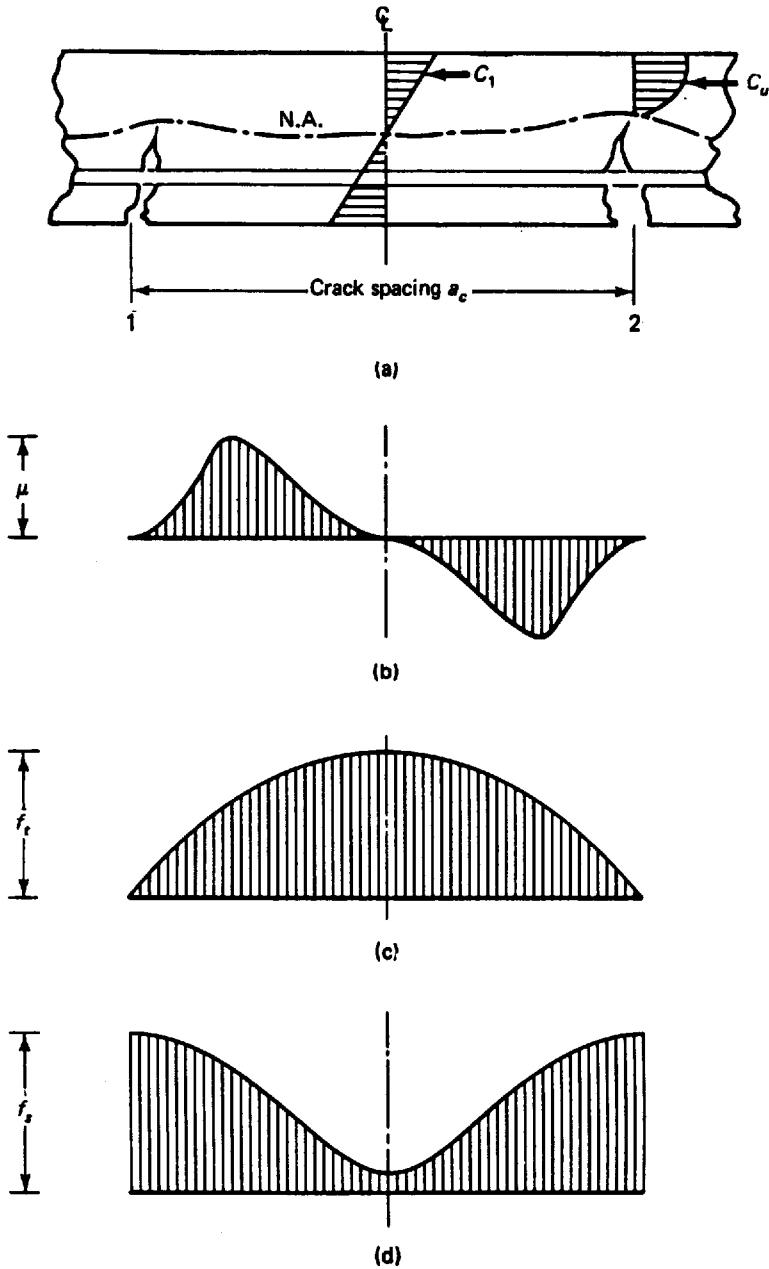


Fig. 2--Schematic stress distribution between two flexural cracks

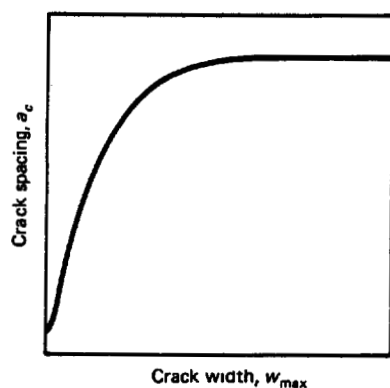
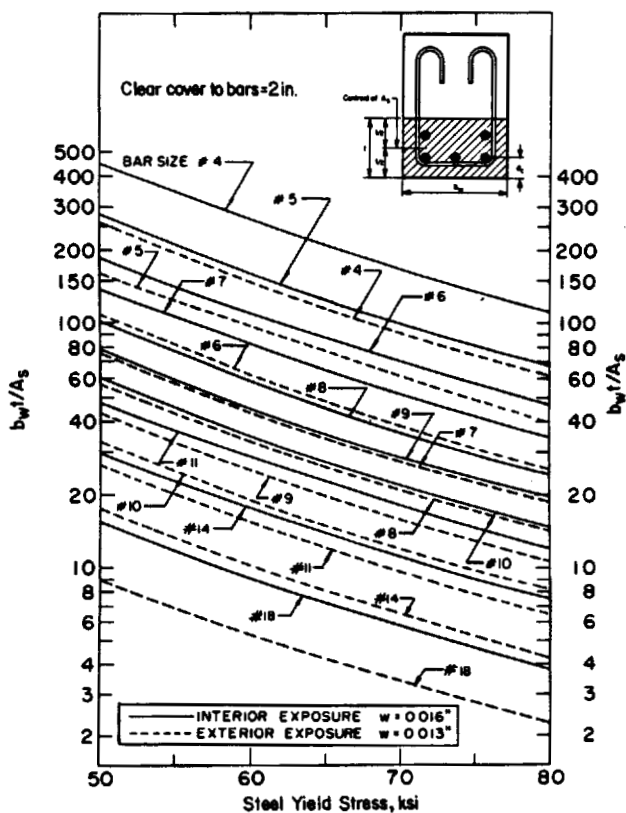


Fig. 3--Schematic variation of crack width with crack spacing

Fig. 4--Steel reinforcement strength f_y versus ratio of concrete area in tension to reinforcement area for stress level $f_s = 0.6 f_y$ (ksi)

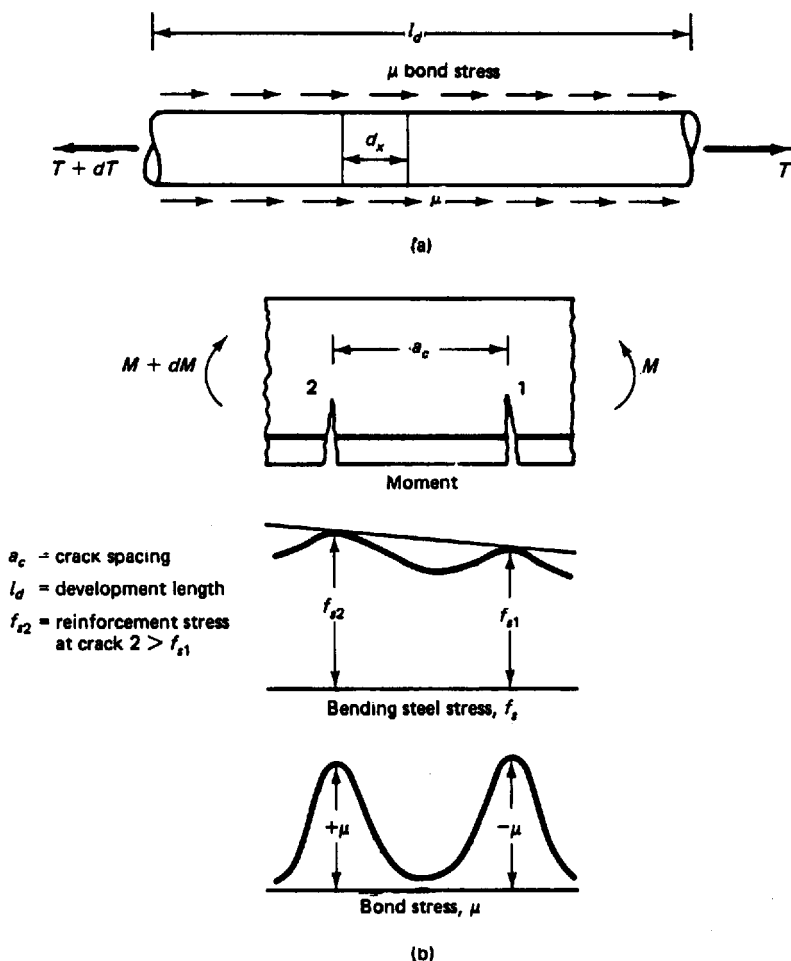
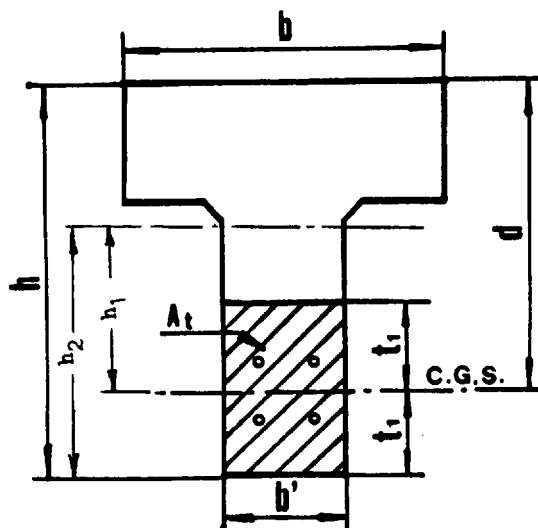
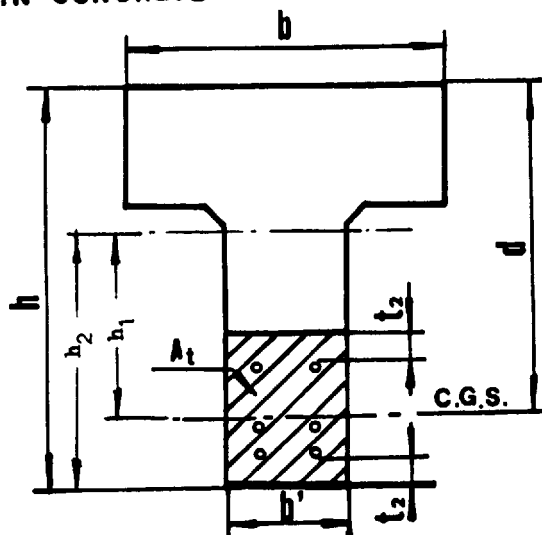


Fig. 5--Force and stress distribution in a stabilized crack of a prestressed beam



(a) FOR EVEN DISTRIBUTION OF REINFORCEMENT
IN CONCRETE



(b) FOR NON-EVEN DISTRIBUTION OF REINFORCEMENT
IN CONCRETE

Fig. 6--Effective area in tension

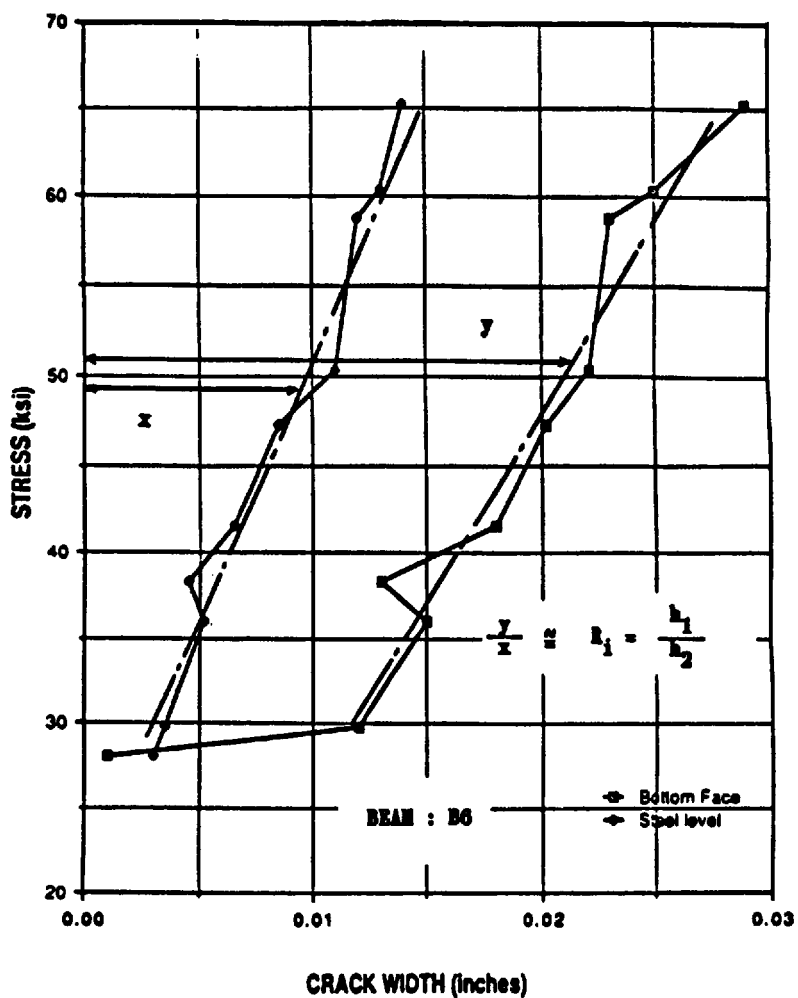


Fig. 7--Typical linear relationship between crack width at steel level and at the concrete tension

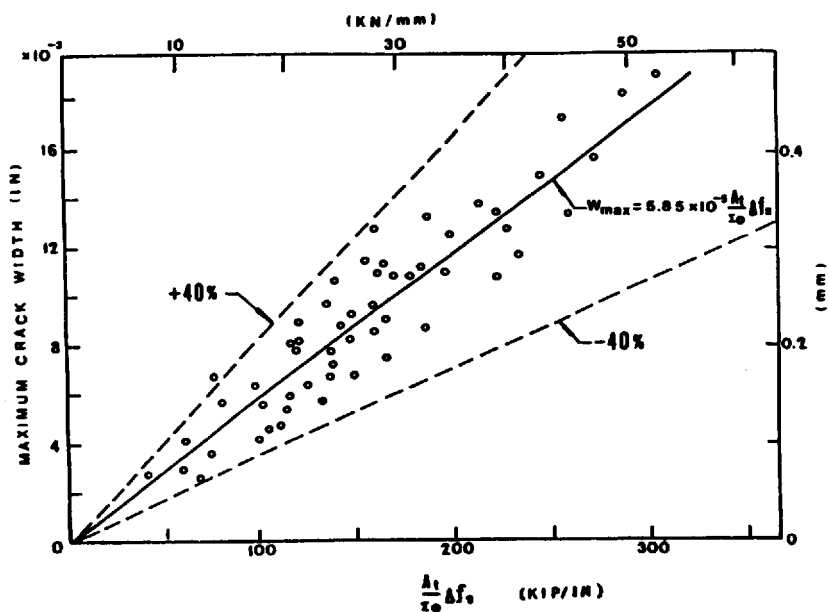


Fig. 8--Linearized maximum crack width versus $(A_t / \Sigma\sigma_o) / \Delta f_s$ (pretensioned beams)

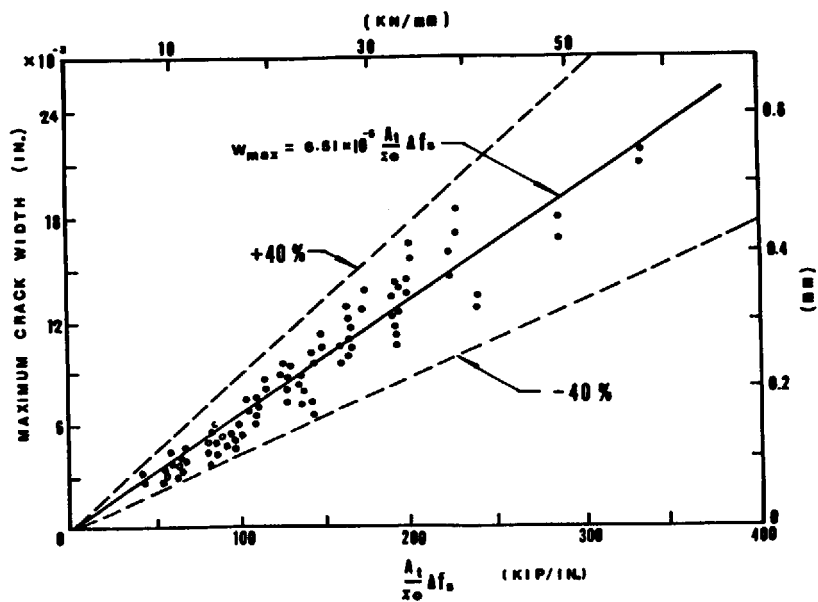


Fig. 9--Linearized maximum crack width versus $(A_t / \Sigma\sigma_o) / \Delta f_s$ (post tensioned beams)

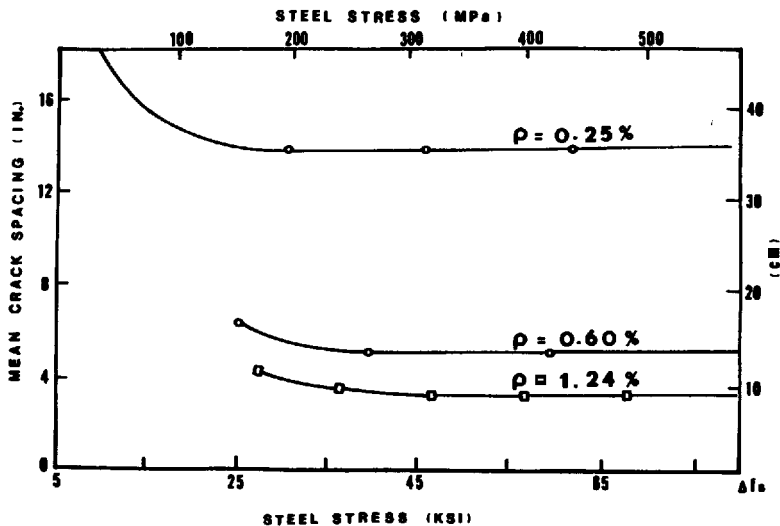


Fig. 10--Effect of steel percentage on mean crack spacing in prestressed beams

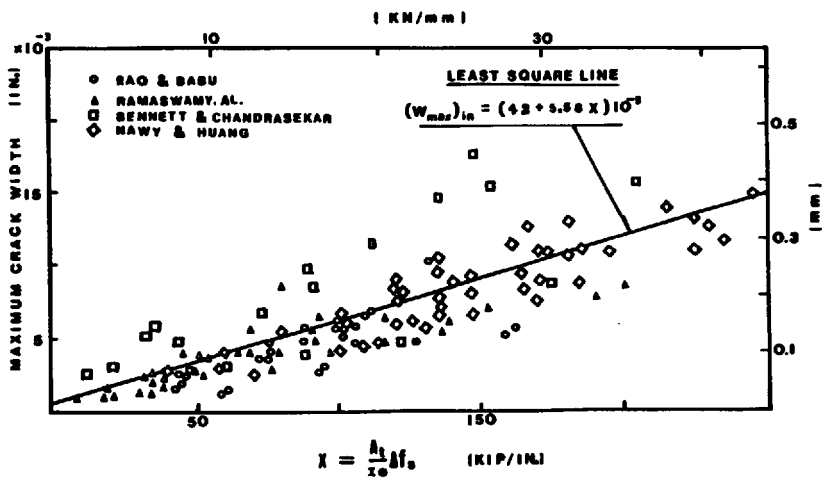


Fig. 11--Reinforcement stress versus crack width (best fit data of several investigators)

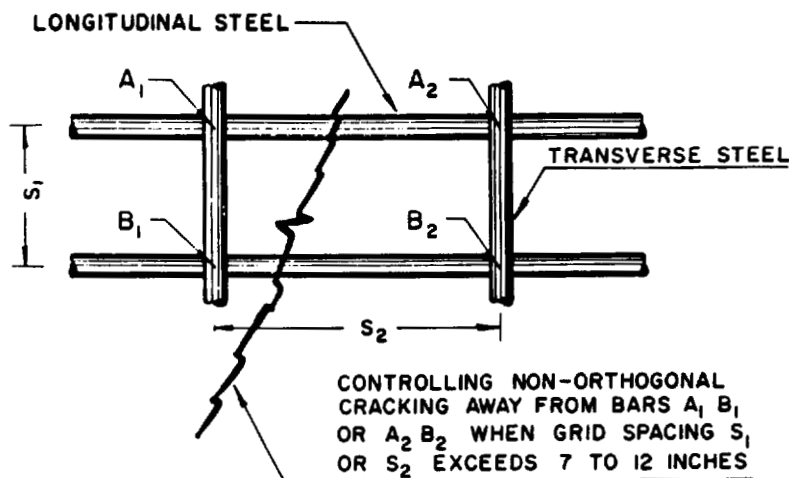
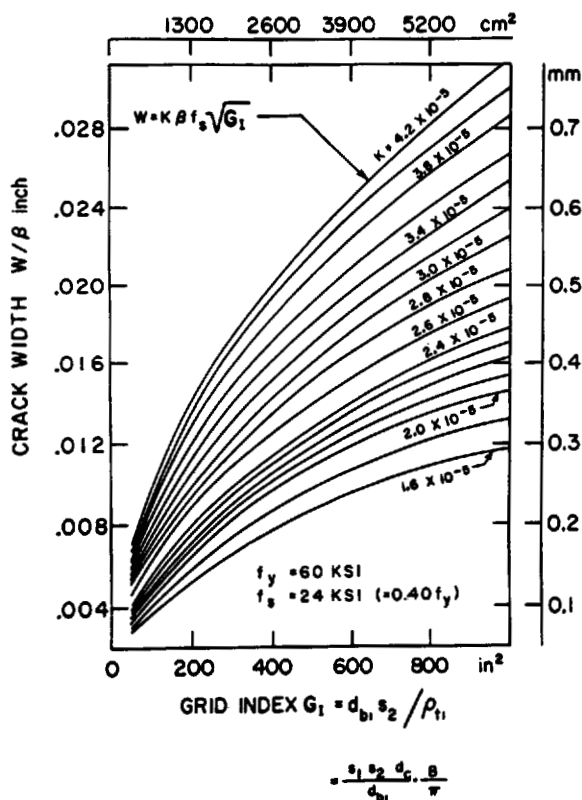


Fig. 12--Grid unit in two-way-action reinforcement

Fig. 13--Crack control reinforcement distribution in two-way-action slabs and plates for all exposure conditions: $f_y = 60 \text{ ksi}$, $f_s = 24 \text{ ksi } (= 0.40 f_y)$

SP 133-2

Safety Provisions in Design Codes for Reinforced Concrete Structures

by A. Scanlon and R.B. Corotis

Synopsis: The American Concrete Institute Standard 318, Building Code Requirements for Reinforced Concrete, has permitted the design of reinforced concrete structures in accordance with limit state principles using load and resistance factors since 1963. A probabilistic assessment of these factors and implied safety levels is made, along with consideration of alternate factor values and formats. A discussion of issues related to construction safety and safety of existing structures is included.

Keywords: Building codes; buildings; limit state design; loads (forces); probability theory; reinforced concrete; safety; structural design; structural engineering; theories

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INTRODUCTION

Working stress principles and linear elastic theory formed the basis for reinforced concrete design prior to 1963, when the concept of ultimate strength design was incorporated in the ACI Building Code (ACI 318-63) [1]. Because of the highly nonlinear nature of reinforced concrete behavior, the linear approach was unable to provide a realistic assessment of true safety levels. This was recognized by the developers of ACI 318-63, who introduced the idea of load and resistance factors to account for uncertainties in both load and resistance (or strength).

While probabilistic methods for code calibration were not highly developed in the early 1960s, the load factors were set with the idea that the probability of exceeding the factored gravity loads would be approximately 0.001, while the strength reduction factors were established to provide a factored strength approximately equal to the 0.01 fractile of the actual strength as illustrated in Fig. 1. These probabilities were entirely judgmental since no substantiating data were available at the time the load and strength reduction factors were set for the 1963 ACI Code [2].

Probabilistic methods were developed and refined during the late 1960s in response to the need to consider variability and uncertainty, explicitly and rationally [3,4]. Proposed formulations included code incorporation of explicit second moment probabilistic procedures [5,6]. In such an approach, the designer would select a desired safety index " β " (Fig. 2) and carry out the design utilizing the means and standard deviations of the load and resistance variables. The safety index positions the mean strength a sufficient number of standard deviations above the mean load effect to ensure attainment of the target reliability. The explicit second moment approach was not considered by ACI 318 or other major code-writing organizations.

CURRENT ACI-318 SAFETY PROVISIONS

The load factors currently in use [7] incorporate some probabilistic considerations, although a knowledge of probability theory is not required of the designer. The basic combination without lateral load is

$$U = 1.4D + 1.7L \quad (1)$$

in which D and L are the dead and live load, respectively. The higher load factor for live load reflects its higher variability. The basic combination involving wind load, W , is

$$U = 0.75(1.4D + 1.7L + 1.7W) \quad (2)$$

The load combination factor, 0.75, is intended to produce a combined value with a reasonable probability of occurrence, since it is extremely unlikely that the design level live and wind loads would occur at the same time. A flaw in the above representation of combined loads is that there is no rational basis for reducing the dead load which is always acting on the structure.

Strength reduction factors were also selected semi-empirically. Values for each type of failure, such as flexure, shear, compression or bearing, reflect the variability of the controlling parameter, including steel yield strength and concrete compressive strength, as well as consequences of the mode of failure. For instance, ductile failure is usually preceded by more warning signs than brittle failure.

While the current ACI 318 format takes into account relative uncertainties among the loads and resistances, there are a number of possible changes that would improve the consistency and the uniformity of reliability in the Code. These will be discussed in the following section.

POSSIBLE CHANGES TO ACI-318

Three areas are identified where changes to the current ACI safety provisions should be considered. These changes relate to the adoption of ANSI A58.1 load factors, recalibration of ACI strength reduction factors, and adoption of partial resistance factors.

ANSI A58.1-1982 Load Factors

The 1982 edition of the American National Standard, Minimum Design Loads for Buildings and other Structures [8], contains load combinations to be used in checking strength limit states. While the format is similar to that used in ACI 318, there are significant differences. The basic combinations without lateral load are

$$U = 1.4D \quad (3)$$

$$U = 1.2D + 1.6L \quad (4)$$

The basic combination involving wind load is

$$U = 1.2D + 0.5L + 1.3W \quad (5)$$

A comparison of these equations with Eqs. (1) and (2) indicates that in general the load factors in ANSI A58.1 are lower than in ACI 318, and that the load combination factor is handled differently. The ANSI A58.1 load factors were developed based on extensive probabilistic studies incorporating available statistical data on loads [9,10]. Their adoption by ACI and other specification-writing groups would allow a uniform set of load factors for all types of construction. Of course, it would be necessary to adjust the resistance side of the equation in conjunction with adopting new load factors.

Recalibration of ACI Strength Reduction Factors

In the event that ACI 318 were to adopt the ANSI load factors with no change in current format, it would be possible to approximate current levels of safety by appropriate adjustment of the strength reduction factors. A comprehensive illustration of a reliability-based calibration of code format to current acceptable practice has been completed, and a proposal for converting the ACI strength reduction factors to a set consistent with the ANSI load factors has been published [11]. Currently, ACI Committee 318 is considering adapting the ACI strength reduction factors to the common load requirements in ANSI A58.1.

Adoption of Partial Resistance Factors

In the ACI 318 safety format, strength reduction factors are applied to overall strength quantities such as flexural strength and shear strength. The single resistance factor has the same shortcoming that a single load factor would have: failing to recognize the differing degrees of uncertainty inherent in different variables. In the partial resistance factor format, resistance factors are applied to the material characteristics rather than to the overall strength quantity. For example, the factored steel yield strength is given by $\phi_s f_y$ and the factored concrete compressive strength by $\phi_c f_c$. Such an approach leads to a more uniform reliability over a variety of load ratios and member designs [12]. It is also possible to introduce a multiplicative overall resistance factor that is dependent on the mode of failure [13]. This has the advantage of being able to introduce mode-specific information, such as the level of uncertainty in the model used to determine strength. This may be used, for example, to reflect the fact that equations for predicting shear strength are generally less reliable than those used to determine flexural strength of under-reinforced cross-sections.

Partial factors can also be applied to account for uncertainties in dimensions. The standard deviation of dimensional variables tends to be relatively constant over a wide range of specified dimensions. The partial factor for dimensions would logically be of an additive form, i.e. $d - \delta_d$.

Using this approach, a general design equation would be of the form

$$\gamma(\gamma_D D + \gamma_L L + \gamma_W W) < \phi R (\phi_s f_y, \phi_c f_c, d - \delta_d) \quad (6)$$

Illustrative Example

To evaluate alternative code formats, Israel et al. [14] designed typical representative beams and columns in accordance with three different formats. The first format is the current ACI 318-83 code; the second is the proposed revision to the code by MacGregor [11], in which ANSI A58-1982 load factors are introduced along with commensurate changes in the overall resistance factors; and the third uses current ANSI load factors with partial resistance factors (without overall factors). Only gravity load cases were considered; consideration of wind forces would be necessary before criteria could be finalized. The following is a summary of the results obtained for flexure, shear, and compression.

For flexural analysis of under-reinforced beams ranging in depth from 18 in. (0.46 m) to 34 in. (0.86 m), the ACI method gave reliability indices varying from 3.01 to 3.78, the proposed revision (with an overall factor of $\phi = 0.85$) gave values from 2.88 to 3.32, and the partial factor format (with $\phi_s = 0.9$, $\phi_c = 0.6$, and $\delta_d = 0.5$ in. = 13 mm) gave values from 2.99 to 3.37. Moreover, the partial factor results were insensitive to variations in steel reinforcement ratio from 20% to 50% of the balanced steel ratio. A significant possibility of compressive failure existed for steel ratios above 65% of the balanced ratio, ρ_b , and consideration might be given to decreasing the current allowable value of $0.75 \rho_b$.

The ACI provisions for shear, including limits on maximum stirrup spacing, gave reliability index values ranging from 3.20 to 5.07. Values for the proposed revision varied from 3.16 to 5.03 ($\phi = 0.75$), while those from the partial factor format varied from 3.39 to 5.02 (using the same partial resistance factors as with the flexural analysis).

Columns analyzed had square cross sections of 14 in. (0.36 m) and 18 in. (0.46 m); eccentricity ratios, e/h , from 0.0 to 1.25 e_p/h ; and gross reinforcement ratios of 0.01 to 0.03. The current ACI provisions gave reliability index values from 2.96 to 4.70, the proposed method gave values from 2.75 to 4.47 ($\phi = 0.65$), and the partial factor method gave values from 3.08 to 4.07 (same factors as above).

From this study, it appears that partial factors of $\phi_s = 0.9$, $\phi_c = 0.6$, and $\delta_d = 0.5$ in. (13 mm) make it possible to attain reliability indices in a relatively narrow range over a wide range of possible member design situations.

SAFETY DURING CONSTRUCTION AND SAFETY OF EXISTING STRUCTURES

In this section a brief overview is given of two areas that are receiving increasing attention in discussions of code safety provisions. While no specific recommendations are given for code implementation, the need for additional research is highlighted along with issues that need to be addressed.

Safety During Construction

While the scope of ACI 318 includes both design and construction, the safety requirements of Chapter 9 address only those load and strength

requirements that pertain to the service life of the structure. No specific guidance is given with regard to the construction phase, often the most critical time in the life of the structure. This is understandable given the complex nature of the construction process and the difficulty of setting specific rules that can be applied to a wide range of construction procedures. However, since the majority of structural failures occur during construction, there is clearly a need to address the issue of expanding the current safety requirements to include the construction phase. Lew [15] discussed the need to consider construction loads in the design of formwork, shoring, and re-shoring, as well as effects on the partially completed structure.

Reliability methods can, in principle, be applied to the construction phase. However, attention must be paid to the strong dependence on time of the development of load and resistance of a concrete structure as construction proceeds. Safety checks of the structure in service are made on the completed structure. During construction, safety depends on the strength and stability of the partially completed structure and on the adequacy of temporary support systems. In addition, if the strength of the structure is strongly dependent on concrete strength, the time dependence of concrete strength should be taken into account.

Safety of Existing Structures

Another area of growing importance is the safety evaluation of existing structures. Chapter 20 of ACI 318 applies to safety evaluation using both load tests and analytical methods. In the load test provisions, a test load of 85% of the factored design load is to be applied for a 24-hour period. Performance is evaluated in terms of the deflection sustained during the load test and the deflection recovery after removal of the test load. The basis for the code provisions is not well documented.

The requirements for analytical evaluation are general in nature and emphasize that the actual condition of the structure should be considered in the evaluation. It has been suggested [16] that probabilistic concepts can be applied to existing structures and that by appropriate sampling of the in-situ condition, the level of uncertainty associated with the existing structure can be assessed and incorporated into the strength evaluation.

Uncertainties that have to be accounted for in satisfying safety requirements for new structures are different than those associated with existing structures. For new structures, the variability measures are those for structures in general and the designer cannot know at the design stage what the actual strength will be. On the other hand, for an existing

structure, an engineer can in principle determine to any desired level of certainty the actual load carrying capacity. This can be achieved by collecting and analyzing various amounts of data from the structure and in the final analysis by performing a load test to failure. Of course, a load test to failure will be of little value if one is interested in using the structure in the future.

Hall [17] has suggested that one can use the successful service record of an existing structure to assess its current safety level. If data are available on existing or past loads, it is suggested that an updated, truncated distribution of strength can be determined using these data as a proof load test. In other words, successful past performance can be combined with a first estimate of strength to provide an improved estimate of structural resistance. The procedure suggested by Hall applies to nondeteriorating structures.

Another approach is to collect data from the structure and to use these data to develop distribution functions for the various parameters that enter into resistance calculations. These distributions can then be used explicitly in probability based limit states analyses similar to those used in developing load and resistance factors for design codes. Questions that have to be addressed include selection of appropriate sample sizes, incorporating errors due to sample size, and use of indirect measures such as nondestructive tests to estimate concrete strength (Scanlon and Mikhailoviky [18]).

A Bayesian statistics approach appears to be useful for this type of analysis because one may evaluate the effect of new data in updating the estimates of the statistical parameters required (Kryviak and Scanlon [19]). An example of Bayesian updating is illustrated in Fig. 3. Based on existing knowledge, a "prior" distribution of concrete strength is available. By taking test samples from the structure, an updated distribution can be obtained. A promising application is in the design of data collection strategies to optimize the amount and type of data to be collected. Another advantage of a Bayesian approach is that subjective data may be incorporated in a rational way. Madsen and Lind [20] describe an application of the Bayesian approach to evaluation of strength of series-produced structures.

As mentioned previously, design codes for new structures do not attempt to include effects of gross error during design, construction, or operation in specifying load and resistance factors. Gross errors may be detected in existing structures as a result of in-situ investigation. These errors can then be taken into account in assessing the safety of the structure. The inclusion of human error in structural reliability has been discussed by a number of authors, including Nowak [21] and Lind [22].

A methodology for strength evaluation of existing bridge structures using modified load and resistance factors has recently been proposed in Canada [23].

CONCLUSION

For about 30 years, the reinforced concrete industry has been a leader in applying the load factor design method, which has permitted a reasonably consistent level of safety for a complicated, composite material. During this time period probabilistic methods have been developed and statistical data on loads and resistances have been accumulated. This information can be incorporated to further improve the safety provisions in the code from the point of view of consistency and uniformity of safety. With the development of the ANSI load factors there is now an opportunity to develop uniform safety formats applicable to all construction materials. Finally, it would be desirable to extend the probabilistic safety concepts to the areas of safety during construction and safety of existing structures.

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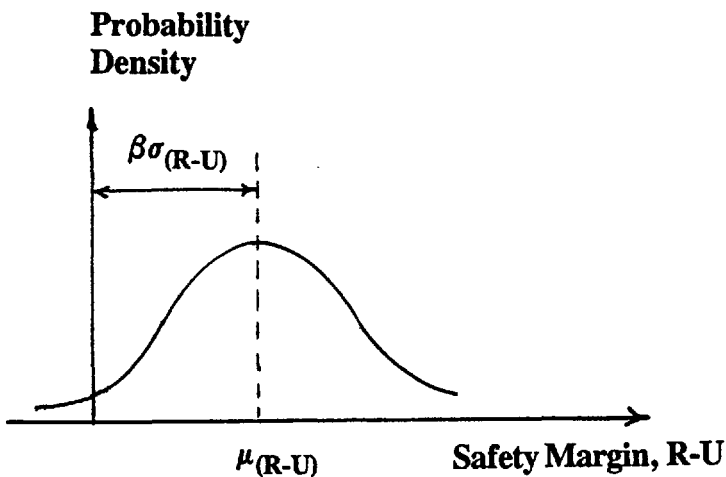


Fig. 1--Probability density function for load and resistance