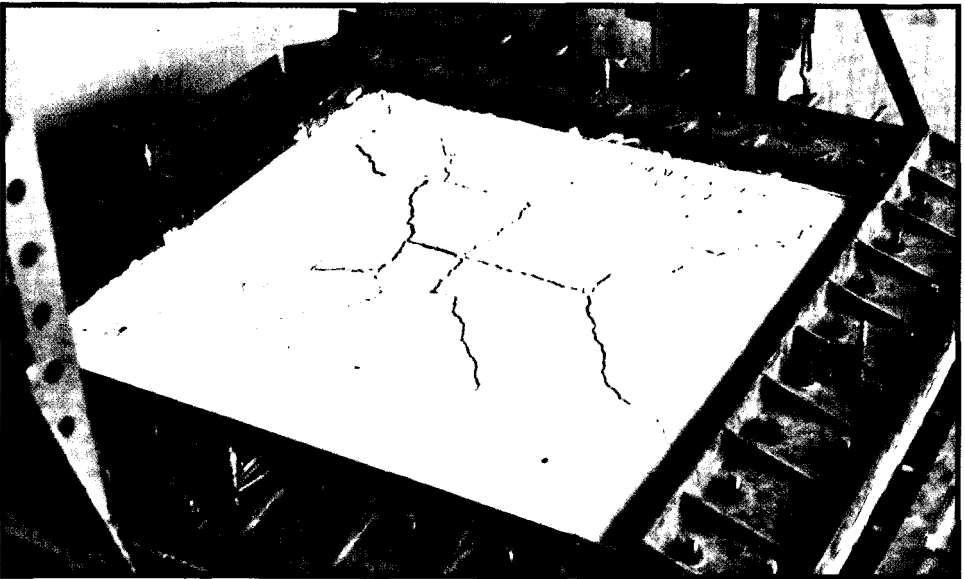


DESIGN AND CONSTRUCTION PRACTICES TO MITIGATE CRACKING



EDITOR: EDWARD G. NAWY
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Design and Construction Practices to Mitigate Cracking

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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 December 2001. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the March/April 2002 issue of either ACI Structural Journal or ACI Materials Journal depending on the subject emphasis of the individual paper.

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PREFACE

Correct design of concrete structural systems requires consideration of serviceability requirements, as determined by crack-control measures. Higher-strength reinforcement, higher-strength concrete, more slender concrete elements, use of a host of admixtures, and emerging construction techniques have increased the need for serious consideration of crack mitigation and crack control in concrete structural systems.

This volume is derived from a national symposium of the American Concrete Institute sponsored by ACI Committee 224, Cracking. The majority of the papers were presented in two highly attended technical sessions in San Diego, CA, entitled "Design and Construction Practices to Mitigate Cracking." The first session was chaired by Edward G. Nawy, and the second session was jointly chaired by Grant T. Halverson and Harvey H. Haynes. The topics in this volume encompass a wide range of subjects, including a detailed summary of worldwide provisions for crack control in reinforced and prestressed concrete beams; two-way slabs and circular tanks, together with the latest Eurocode provisions, including design examples; early-age thermal cracking; diagonal cracking, including seismically induced diagonal cracks; crack mitigation effects of shrinkage reducing admixtures fibers; repair of cracks; cracking in water-retaining structures; and an overview of the cracking developed in the 1999 earthquake in Turkey. This special publication also includes a list of references at the end of each paper, which can be helpful to design engineers and constructors.

All papers presented in this publication were reviewed by recognized experts in accordance with the ACI review procedures. Each paper had a minimum of two reviewers. It is hoped that designers, constructors, and codifying bodies will be able to draw on the vast material presented in this volume along with the recently revised and updated ACI 224 committee report "Control of Cracking in Concrete Structures," in improving the long-term cracking behavior and performance of concrete-constructed facilities.

Edward G. Nawy
Editor and Founding Chairman, ACI Committee 224

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Co-editor and Current Chairman, ACI Committee 224

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Design for Crack Control in Reinforced and Prestressed Concrete Beams, Two-Way Slabs and Circular Tanks – A State-of-the-Art

by E. G. Nawy

Synopsis: This paper presents the state-of-the art in the evaluation of the flexural crack width development and crack control of flexural cracks in reinforced and prestressed concrete structures. 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 elements.

Mitigation and control of cracking has become essential in order to maintain the integrity and aesthetics of concrete structures and their long-term durability performance. The trend is stronger than ever towards better utilization of concrete strength, use of higher strength concretes in the range of 12,000-20,000 psi and higher compressive strength, more prestressed concretes and increased uses of limit failure theories - all these trends require closer control of serviceability requirements of cracking and deflection behavior.

The paper discusses and presents common expressions for the mitigation and control of cracking in reinforced concrete beams and thick one-way slabs, prestressed, pretensioned and post-tensioned flanged beams, reinforced concrete two-way action structural floor slabs and plates, and large diameter circular tanks. In addition, recommendations are given for the maximum tolerable flexural crack widths in concrete elements based on the cumulative experience of many investigators over the past five decades. The expressions include the ACI 318-99 crack control provisions in reinforced concrete beams and one-way slabs, and the Concrete Euro Code 1999 for the design of concrete buildings.

Keywords: beams; concrete; concrete strength; crack control; cracking; crack width; environment; equations for reinforced and prestressed beams; Eurocode; flexural crack width; long-term cracking; tanks; tolerable crack widths; two-way action structural slabs

Edward G. Nawy, F.ACI, is Professor of Civil Engineering, Rutgers University, and holds the distinguished professor rank. Active in ACI since 1949, Professor Nawy is the founding chairman and a current member of ACI Committee 224 on Cracking; past chairman of ACI Committee 435 on Deflection of Concrete Building Structures; member of ACI Committee 340, Design Aids for ACI 318 Building Code; member of Joint ASCE-ACI Committee 421 on Design of Reinforced Concrete Slabs. Professor Nawy has published in excess of 160 papers and is the author four major textbooks and one handbook: SIMPLIFIED REINFORCED CONCRETE (1987); REINFORCED CONCRETE - A fundamental Approach (4th Ed., 2000) and translated into several languages; PRESTRESSED CONCRETE - A Fundamental Approach (3rd Ed., 2000); FUNDAMENTALS OF HIGH PERFORMANCE CONCRETE (2nd Ed., 2001); and CONCRETE CONSTRUCTION ENGINEERING HANDBOOK (1998), as well as chapters in several handbooks. He holds several honors including the ACI Chapter Activities Award, the Henry L. Kennedy Award and the Concrete Research Council's Robert Philleo Award, was twice president of the ACI New Jersey Chapter, served two terms on the Rutgers University Board of Governors, and is Honorary professor, Nanjing Institute of Technology, China. He is a registered professional engineer in New York, New Jersey, Pennsylvania, California and Florida, and a chartered civil engineer overseas. He has also served as program evaluator for the national Accreditation Board for Engineering and Technology (ABET).

INTRODUCTION

Presently, the trend is stronger than ever in better utilization of concrete strength, use of higher strength high-performance concretes of 20,000 psi (138 MPa) compressive strength and higher, use of high-strength reinforcement, more prestressed concretes and increased use of limit failure theories - all these trends require closer control of serviceability requirements in cracking and deflection behavior. Hence, knowledge of the cracking behavior of concrete elements, and how to mitigate cracking, become 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 drying shrinkage, creep under sustained load, thermal stresses including elevated temperatures, and chemical incompatibility of concrete components.
2. Direct stress due to applied loads or reactions or internal stress due to continuity, reversible fatigue load, long-term deflection, camber in prestressed systems, or environmental effects including differential movement in structural systems.
3. Flexural stress due to bending.

While the net result of these three actions can be the formation of cracks, the

mechanisms of their development cannot be considered identical. Volumetric change generates internal micro-cracking which may develop into full cracking, while direct internal or external stress or applied loads and reactions could either generate internal macro-cracking, such as in the case of fatigue due to reversible load, or flexural micro-cracking leading to fully developed cracking.

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

MICRO-CRACKING

Micro-cracking may be mainly classified into two principal categories: a) bond cracks at the aggregate-mortar interface, b) paste cracks within the mortar matrix. Interfacial bond cracks are caused by interfacial shear and tensile stresses due to early volumetric change without the presence of external load. Volume change caused by hydration and shrinkage can create tensile and bond stresses of sufficient magnitude as to cause failure at the aggregate-mortar interface.¹⁻⁶ As the external load is applied, mortar cracks develop due to increase in compressive stress, propagating continuously through the cement matrix up to failure.

It appears that 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 research 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 AND CRACKING MITIGATION

External load causes in direct and bending stresses potentially resulting in flexural, bond and diagonal tension cracks. Once the tensile stress in the concrete exceeds its tensile strength, internal micro-cracks form. These cracks develop into macro-

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cracks propagating towards the external fiber zones of the element.

Immediately after the full development of the first crack in a conventionally reinforced concrete element, the stress in the concrete at the cracking zone is reduced to zero and is assumed by the reinforcement⁵. The distribution of ultimate bond stress, longitudinal tensile stress in the concrete and longitudinal tensile stress in the steel can be schematically represented in Fig. 1.

Crack width is a primary function of the bond characteristics and deformation of reinforcement between the two adjacent cracks 1 and 2 in Fig. 1, if the small concrete strain along the crack interval a_c is neglected. Hence, the crack width would be a function of the crack spacing and vice versa up to the level of stabilization of crack spacing (Fig. 2).

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

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

where w = maximum crack width, and

α , β and γ are nonlinearity constants. Crack spacing a_c is a function of the factors enumerated previously, being inversely proportional to bond strength and active steel ratio (steel percentage in terms of the concrete cross-sectional area in tension). ϵ_s is the reinforcement strain induced by external load.

The basic mathematical model in equation (1) with the appropriate experimental values of the constants α , β and γ can be derived for a particular type of structural member. Such a member can be a one-dimensional element such as 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 forms or expressions apply for the evaluation of the macro-cracking behavior of different structural elements consistent with their fundamental structural behavior.²⁻⁴

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 (span/thickness ratio about 15) in the ACI 318 Building Code are based on the statistical analysis of maximum crack width data from a number of sources. On the basis of this analysis and the vast amount of data available, the following general conclusions were reached:

1. The steel stress is the most important variable.
2. The thickness of the concrete cover is an important variable, but not the only geometric consideration.
3. The area of concrete surrounding each reinforcing bar is also an important geometric variable.
4. The bar diameter is not a major variable.
5. The size of the bottom 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 Eq. 2 as follows⁴ :

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

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.

β = h_2/h_1 where h_1 = distance from neutral axis to the reinforcing steel, in.

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

When the strain, ϵ_s , in the steel reinforcement is used instead of stress, f_s , Eq. (2) becomes

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$$w_{\max} = 2.2 \beta \varepsilon_s \sqrt[3]{d_c A} \cdot 10^{-3} \quad (3)$$

and is valid in any system of measurement.

The cracking behavior in thick one-way slabs, namely those with clear cover exceeding 1-1/2 in., is similar to that in shallow beams. For such one-way slabs (Eq. 3) can be adequately applied if β ranging from 1.25 to 1.35 is used.

ACI 224 Report³ and ACI 340 Report give recommendations for tolerable crack widths under various environmental conditions, as given in Table 1.

ACI 318-99 Code Provisions

The ACI 318-99 mitigates cracking through control of the spacing of the reinforcing bars,^{1, 12, 13} as an indirect measure of crack control. The expression that the code requires to be used is:

$$s \text{ (in.)} = (540 / f_s) - 2.5 c_c \quad (4)$$

but not greater than $12(36 / f_s)$, where

f_s = computed stress in the reinforcement at service load = unfactored moment divided by the steel area and the internal arm moment. Alternatively, f_s can be taken as $0.60 f_y$.

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

s = center-to-center spacing of flexural tension reinforcement, inches, closest to the tension face of the section.

From these provisions, the maximum spacing for 60,000 psi (414 MPa) reinforcement = $12 [36 / (0.6 \times 60)] = 12$ in. (305 mm). The maximum allowable spacing of 12 inches is in conformity with the extensive testing performed by the author on over excess 100 two-way action slabs, discussed in subsequent sections. Hence this limitation on the distribution of flexural reinforcement in one-way slabs and wide-web reinforced concrete beams is appropriate. However, in beams of normal web width in usual buildings, these provisions might not be as workable.¹⁴ In

the opinion of the author, it is more advisable and perhaps safer to use the applicable crack width equation in designing for the tolerable crack width that the particular environmental condition requires.

The SI expression for the value of reinforcement spacing in Eq. 4 and f_s in MPa units is,

$$s \text{ (mm)} = (95,000 / 540 f_s) - 2.5 c_c \quad (5)$$

but not to exceed $300(252/f_s)$. For the usual case of beams with grade 420 reinforcement and 50 mm clear cover to the main reinforcement, with $f_s = 252$ MPa, the maximum bar spacing is 250 mm.

It should be noted that if the concrete structural member is in severe environment, where the ACI bar spacing provisions can result in crack widths of 0.0137 in. or more, satisfying the ACI 318-99 requirements for bar spacing is not adequate for sustaining the long-term structural integrity of the member. This is why, use of the Gergely-Lutz equation in conjunction with Table 1, or the Euro EC2 Code expressions presented in the next section, is the advisable design route to follow for crack control in beams and thick one-way slabs.

Euro EC2 Code Provisions

The Euro Code EC2 requires that cracking should be limited to a level that does not impair the serviceability of the structure or cause its appearance to be unacceptable.⁹ It limits the maximum design crack width to 0.3 mm (0.012 in.) under normal environmental conditions and under quasai permanent combination of loads. This ceiling is expected to be satisfactory with respect to appearance and durability. Stricter requirements of tolerable crack width are stipulated for more severe environmental conditions.

The EC2 Code stipulates that the design crack width be evaluated from the following expression:

$$w_k = \beta s_{rm} \epsilon_{sm} \quad (6)$$

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where

w_k = design crack width

s_{rm} = average stabilized crack spacing

ϵ_{sm} = mean strain under relevant combination of loads and allowing for the effect of tension stiffening, shrinkage, etc.

β = coefficient relating the average crack width to the design value

= 1.7 for load-induced cracking and for restraint cracking in sections with minimum dimension in excess of 800 mm (32 in.

The strain, ϵ_{sm} , in the section is obtained from the following expression:

$$\epsilon_{sm} = \sigma_s / E_s [1 - \beta_1 \beta_2 (\sigma_{sr} / \sigma_s)^2] \quad (7)$$

where

σ_s = stress in the tension reinforcement computed on the basis of a cracked section.

σ_{sr} = stress in the tension reinforcement computed on the basis of a cracked section under loading conditions that cause the first crack.

β_1 = coefficient accounting for bar bond characteristics

= 1.0 for deformed bars and 0.5 for plain bars.

β_2 = coefficient accounting for load duration

= 1.0 for single short-term loading and 0.5 for sustained or cyclic loading

E_s = Modulus of elasticity of the reinforcement

The average stabilized mean crack spacing, s_{rm} , is evaluated from the following expression:

$$s_{rm} = 50 + 0.25 k_1 k_2 d_b / p_t, \text{ mm} \quad (8)$$

where

d_b = bar diameter, mm.

p_t = effective reinforcement ratio = A_s / A_t ; the effective concrete area in tension, A_t , is generally the concrete area surrounding the tension reinforcement, of depth equal to 2.5 times the distance from the tensile face of the concrete section to the centroid of the reinforcement. For slabs where the depth of the tension zone may be small, the height of the effective area should not be taken less than $(c - d_b) / 3$, where c = clear cover to the reinforcement.

k_1 = 0.8 for deformed bars and 1.6 for plain bars.

k_2 = 0.5 for bending and 1.0 for pure tension.

In cases of eccentric tension or for local areas, an average value of $k_2 = (\epsilon_1 + \epsilon_2) / 2$ ϵ_1 can be used, where ϵ_1 is the greater and ϵ_2 the lesser tensile strain at the section boundaries, determined on the basis of cracked section.

In the absence of rigorous computations as described thus far, choice of minimum area of reinforcement, A_s , for crack control is stipulated such that

$$A_s = k_{ck} f_{ct,eff} A_{ct} / \sigma_s \quad (9)$$

where

A_s = reinforcement area within the tensile zone.

A_{ct} = Effective area of concrete in tension

σ_s = maximum stress permitted in the reinforcement after the formation of the crack. The yield strength may be taken in lieu of σ_s , although lower values may be needed to satisfy crack width limits.

$f_{ct,eff}$ = tensile strength of the concrete effective at the formation of the first crack. A value of 3 N/mm² (435 psi) can be used.

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k_c = coefficient representing the nature of stress distribution,

= 1.0 for direct tension and 0.4 for bending

k = coefficient accounting for non-uniform stresses due to restraint resulting from intrinsic or extrinsic deformation. It varies between 0.5 and 1.0.

The EC2 Code also stipulates that for cracks dominantly caused by flexure, their width will not usually exceed the standard 0.30 mm (0.012 in.), if the size and spacing of the reinforcing bars are within the range of values in Tables 2 and 3 for bar size and spacing. Evidently, for severe exposure conditions, such as those listed in Table 1 for tolerable crack widths beyond the 0.012 in. crack width level, crack width computations become mandatory.

The Australian Code Provisions on Flexural Crack Control

The Australian Code does not recommend any formula for the calculation of crack widths. Crack control for flexure in reinforced concrete beams is achieved if the center-to-center spacing of bars near the tension face of the beam does not exceed 8 in. (200 mm) and the distance from the side or soffit to the center of the nearest longitudinal bar is not greater than 4 in. (100 mm). In the case of fully prestressed concrete beams, the maximum tensile stress in the concrete due to short-term service loads should not exceed $3\sqrt{f'_c}$. To control flexural cracking in partially prestressed concrete beams, the increment in steel stress near the tension face is limited to 29Ksi (200 MPa), as the load increases from its value when the extreme concrete tensile fiber is at zero stress to the short-term service load value; and the center-to-center spacing of reinforcement, including bonded tendons, is limited to 8 in. (200 mm).

Flexural cracking in reinforced concrete slabs is controlled by limiting the center-to-center spacing of bars in each direction to the lesser of 2.5 times the thickness of slab or 20 in (500 mm). In fully prestressed slabs, similar to beams, the maximum tensile stress in the concrete due to short-term service loads is limited to $3\sqrt{f'_c}$. For partially prestressed slabs, the incremental steel stress should not exceed 22 ksi (150

MPa) and the center-to-center spacing of reinforcement including bonded tendons, is not to exceed 20 in. (500 mm). It should be noted that the extensive Nawy work⁵⁻⁷ demonstrated that the maximum crack spacing in two-way reinforced concrete slabs should not exceed 12 in. (300 mm), otherwise yield line wide cracks would be prematurely generated. Hence, the ACI 318 Code limits the maximum spacing to twice the slab thickness.

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

The increased use of partial prestressing, allowing limited tensile stresses in the concrete under service and overload conditions while allowing non-prestressed steel to carry the tensile stresses, is becoming prevalent due to practicality and economy. Consequently, an evaluation of the flexural crack widths and spacing and control of their development 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, have undertaken extensive research since the 1960's on the cracking behavior of prestressed pretensioned and post-tensioned beams and slabs because of the great vulnerability of the highly stressed prestressing steel to corrosion and other environmental effects and the resulting premature loss of prestress.^{10,11} Serviceability behavior under service and overload conditions can be controlled by the design engineer through the application of the criteria presented in this section.

A. Mathematical Model Formulation for Serviceability Evaluation

1. 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 and the number of cracks will be stabilized when the stress in the concrete no longer

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exceeds its tensile strength at further locations regardless of load increase. This condition is important as it essentially produces the absolute minimum crack spacing which can occur at high steel stresses, to be termed the stabilized minimum crack spacing. The maximum possible crack spacing under this stabilized condition is twice the minimum, to be termed the stabilized maximum crack spacing. Hence, the stabilized mean crack spacing, a_{cs} , is evaluated as the mean value of the two extremes.

The total tensile force T in Fig. 3 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 $\sqrt{f'_c}$

Σ_o = sum of reinforcing elements' circumferences

The resistance R of the concrete area in tension A_t 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, where c is a constant to be developed from the tests:

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

The concrete stretched area, namely the concrete area, A_t , in tension for both the evenly distributed and non-evenly distributed reinforcing elements, is illustrated in

Fig. 4. 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

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

2. Crack Width

If Δf_s is the net stress in the prestressed tendon or the magnitude of the tensile stress in the normal steel at any crack width load level in which the decompression load (decompression here means $f_c = 0$ at the level of the reinforcing steel) is taken as the reference point, then for the prestressed tendon

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

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

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

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

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

B. Expressions for Pretensioned Beams

Eq. 12a is rewritten in terms of Δf_s so that analysis of the test data of all the simply supported test beams in this work leads to the following expression at the reinforcement level:

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

Linearizing Eq. 13 for easier use by the design engineer leads to the following simplified expression of the maximum crack width at the reinforcing steel level closest to the tensile face of the beam:

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

In SI units, the expression in Eq. 14 (a) becomes

$$w_{\max} (\text{mm}) = 8.48 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (14b)$$

where $A_t = \text{cm}^2$, $\Sigma o = \text{cm}$ and $\Delta f_s = \text{MPa}$.

The maximum crack width (in.) at the tensile face of the concrete is

$$w_{\max} (\text{in.}) = 5.85 \times 10^{-5} R_i \frac{A_t}{\Sigma o} (\Delta f_s) \quad (14c)$$

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

A plot of the data and the best fit expression for Eq. 14a is given in Fig. 5 with a 40 percent spread, which is reasonable in view of the randomness of crack development and the linearization of the original expression of Eq. 13.

C. Expressions for Post-Tensioned Beams

The expression developed for the crack width at the reinforcement level closest to the tensile face in post-tensioned bonded beams which contain mild steel reinforcement

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

In SI units, the expression in Eq. 15(a) becomes

$$w_{\max} (\text{mm}) = 9.44 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (15b)$$

For the crack width at the tensile face of the concrete lower fibers:

$$w_{\max} \text{ (in.)} = 6.51 \times 10^{-5} R_i \frac{A_t}{\sum o} (\Delta f_s) \quad (15c)$$

For non-bonded beams, the factor 6.51 in Eqs. 15a and 15 b becomes 6.83.

A plot of the data and the best fit expression for Eq. 15a is given in Fig. 6.

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. 7. It is seen from this plot that crack spacing stabilizes at a net stress level of 36 to ksi.

D. Cracking of high-strength high performance prestressed beams.

More recent work at Rutgers University on the cracking behavior of high-strength pretensioned and non-bonded post-tensioned beams having cylinder compressive strengths, f'_c , in the range of 10,200 psi to 14,200 psi (70.3 to 97.9 MPa), have resulted in the following expression for the crack width at the reinforcement level of the pretensioned member

$$w_{\max} \text{ (in.)} = 2.74 \times 10^{-5} \frac{A_t}{\sum o} (\Delta f_s) \quad (16a)$$

or, in SI units:

$$w_{\max} \text{ (mm)} = 4.0 \times 10^{-5} \frac{A_t}{\sum o} (\Delta f_s) \quad (16b)$$

The factor 2.74 in Eq. (16a) is an average of values from the statistical expression that resulted in the expressions given in Eqs. 17 and 18. the reduced crack width due to the use of high-strength high performance concrete, is expected in view of the increased bond interlock between the reinforcement and the concrete.

$$\lambda_r = \frac{2}{(0.75 + 0.06\sqrt{f'_c})\sqrt{f'_c}} \quad (17)$$

For post-tensioned beams, the reduction multiplier λ_o is

$$\lambda_o = \frac{1}{0.75 + 0.06\sqrt{f'_c}} \quad (18)$$

where f'_c and the reinforcement stress are in ksi.

E. Cracking in Circular Prestressed Concrete Tanks

Circular prestressed tanks are cylindrical shell elements of very large diameter in relation to their height. Hence, it is possible to treat the wall with respect to flexural cracking in a manner similar to the behavior of two-way action plates. Vessay and Preston¹⁸ modified the Nawy expressions developed for two-way action slabs and plates so that the maximum crack width can be defined as:

$$w_{\max} (in.) = 4.1 \times 10^{-6} \varepsilon_{ct} E_{ps} \sqrt{G_I} \quad (19a)$$

In SI units, the expression becomes

$$w_{\max} (mm) = 0.6 \times 10^{-6} \varepsilon_{ct} E_{ps} \sqrt{G_I} \quad (19b)$$

where E_{ps} is in MPa and the dimensions of all the parameters of the grid index G_I are in millimeters.

ϵ_{ct} = tensile surface strain in the concrete = $(\lambda_t f_p)/(E_{ps})$

f_p = actual stress in the steel

f_{pi} = initial prestress before losses

$\lambda_t = f_p/f_{pi}$

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

d_{b1} = diameter of steel in direction 1

s_1 = spacing of the reinforcement in direction 1 closest to the tensile face

s_2 = spacing of the reinforcement in direction 2

d_c = concrete cover to center of steel, in.

Note that $w_{max} = 0.004$ in. (0.10 mm) should be the maximum crack width for liquid-retaining tanks and sanitary structures.

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

Flexural crack control is essential in structural floors where cracks at service load and overload conditions can have serious performance effect in such structures as office buildings, schools, parking garages, industrial buildings and other floors, where the design service load levels exceed those in normal size apartment building panels and also in all cases of adverse exposure conditions.

A. Flexural Cracking Mechanism and Fracture Hypothesis

Flexural cracking behavior in concrete structural floors under two-way action is significantly different from that in one-way members. Crack-control equations for beams underestimate the crack widths developed in two-way slabs and plates, and do not tell the designer how to space the reinforcement. Cracking in two-way slabs and plates is controlled 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 [3/4 in. (19 mm) for interior exposure], whereas it is a major variable in the crack-control equations for beams. The

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results from 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. 7 stress concentration develops initially at the points of intersection of the reinforcement in the reinforcing bars and at the welded joints of the wire mesh, that is, at grid nodal points, thereby dynamically generating fracture lines along the paths of least resistance, namely, along A_1B_1 , A_1A_2 , A_2B_2 , and B_2B_1 . The resulting fracture pattern is a total repetitive cracking grid, provided that the spacing of the nodal points A_1B_1 , A_2 , and B_2 is close enough to generate this preferred initial fracture grid of orthogonal cracks narrow in width, as a preferred fracture mechanism.

If the spacing of the reinforcing grid intersections is too large, the magnitude of stress concentration and the energy absorbed per unit grid is 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. In a similar manner, one may conclude that the scale effect on two-way-action cracking behavior is insignificant, since the cracking grid would be a reflection of the reinforcement grid if the preferred orthogonal narrow cracking widths develop. 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 small, constant in value of 0.75 in. (20 mm).

For a constant 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 into preferred orthogonal grids.

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.

B. Crack Control Equation

The basic Eq. 1 for relating crack width to strain in the reinforcement is

$$w = \alpha a_c^\beta \epsilon_s^\gamma \quad (20)$$

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. 17 and the statistical analysis of the data of 95 slabs tested to failure^{5,6}, the following crack-control equation emerged:

$$w = K\beta f_s \sqrt{\frac{d_{b1}s_2}{Q_{i1}}} = K\beta f_s \sqrt{G_1} \quad (21)$$

where the quantity under the radical, $G_1 = d_{b1}s_2/Q_{i1}$, is termed the grid index, and can be transferred into

$$G_1 = \frac{s_1s_2d_c}{d_{b1}} \cdot \frac{8}{\pi} \quad (22a)$$

where

K = fracture coefficient, having a value of 2.8×10^{-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

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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 stress level, or 50% of the design yield strength, f_s (ksi).

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

s_1 = spacing of the reinforcement in direction 1.

s_2 = spacing of the reinforcement in perpendicular direction 2.

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

$$Q_{i1} = \text{active steel ratio} = \frac{\text{Reinforcement area } A_s \text{ per unit width}}{12(d_{b1} + 2c_1)} \quad (22b)$$

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 at the service level (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.

Using SI units, the expression in Eq. 21 becomes

$$w_{\max} (mm) = 0.145 K \beta f_s \sqrt{G_I} \quad (23)$$

where f_s is in MPa and all the terms for the grid index G_i in Eq. 21 are in mm.

A graphical solution of Eq. 2 is given in Fig. 8 for rapid determination of the reinforcement size and spacing needed for crack control where $f_y = 60,000$ psi (414 Mpa) and $f_s = 40\% f_y = 24,000$ psi (165.5 MPa).

C. Tolerable Crack Widths in Concrete Structures

The maximum reasonable crack width that can be tolerated in a structural element without distress depends on the particular function of the element and the environmental conditions to which the structure is liable to be subjected. Table 1 from the ACI Committee 224 Report³ 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 tolerable crack widths in Table 1 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, and not exceeding 12 in. 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 in cases of cyclic loading, such as in bridges, but the width increases at a decreasing rate with

time. In most cases, a doubling of crack width or even a larger increase after several years under sustained loading can be expected.

Crack control in deep beams

Several cases have been reported,^{3,19} where wide cracks have developed on the side faces of beams between main flexural reinforcement and the neutral axis. Although the measured crack widths at the main reinforcement level were within acceptable code limits, the side face crack widths near mid-depth were as much as three times as wide.

Based on experimental and analytical investigations of cracking in deep beams, Frantz and Breen developed recommendations for side face crack control in beams in which the depth, d , exceeds 36 in. (915 mm).¹⁹ ACI 318 requires now skin reinforcement to be uniformly distributed along both faces of the member for a distance $d/2$ nearest the flexural tension reinforcement.

Anchorage zone cracking in prestressed concrete

Longitudinal cracks develop in the anchorage zones of prestressed concrete members, particularly in post-tensioned beams, due to transverse tensile stresses set up by the concentrated forces at the end zones. To prevent such cracks from developing, which can lead to failure of the member, transverse reinforcement has to be used applying proper design procedures, such as the strut-and-tie method for proportioning the reinforcement. Two types of cracks may develop: spalling cracks which begin at the end face and propagate parallel to the prestressing force, and bursting cracks which can develop along the line of the force or forces, but away from the end face.

For many years stirrups were designed to take the entire calculated tensile force based on the analysis of the uncracked section. Classical and finite element analyses by Nawy¹⁷ and Breen et al.¹⁸ show similar stress distributions for which confining closed stirrups have to be provided. Also, use of spiral reinforcement indicates more beneficial effect in arresting the end anchorage zone cracks when used as transverse reinforcement.¹⁹ It is advisable that more closely spaced closed stirrups be provided at the end anchorage zones of post-tensioned members than what the theoretical analysis indicates. AASHTO Specifications give detailed guidelines on the

design of anchorage zone reinforcement to prevent bursting and spalling cracks, primarily using the strut-and tie approach for the proportioning and location of the reinforcement.^{11, 20}

Tolerable crack widths in prestressed members

The Euro EC2 Code and some authors state that corrosion is a greater problem in prestressed concrete members because of the smaller area of steel used and because of the possible consequences of corrosion on high stressed steel.^{20, 21, 22} The CEB Model Code,^{3,8} does not allow any tension in the concrete for severe exposure conditions, namely, no cracking is permitted. Poston et al^{3, 21, 22} found that chloride ion concentration at the level of reinforcement due to penetration of chlorides from external sources to be basically proportional to crack width. In general, a crack width not exceeding 0.005 in. (13 mm) is the upper tolerable limit for prestressed members in aggressive environment.

CONCLUSIONS

With the aid of the expressions presented in this paper summarized in the following, the design engineer and the constructor can limit the flexural macro-crack widths that develop in concrete systems. By limiting the width to remain within the tolerable levels given in Table 1 for the prevailing environmental conditions, it would be possible to prevent or considerably minimize long-term corrosion deterioration and deflection increase, and also maintain the aesthetic characteristics of the various elements of the system.

1. Reinforced Concrete Beams and Thick One-Way Slabs:

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

$$s \text{ (in.)} = (540/f_s) - 2.5 c_c, \text{ but not greater than } 12(36/f_s), \text{ where}$$

f_s is in ksi (see Eq.4).

The Euro EC2 Code requires crack width computation for cracking mitigation, using

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the following expression for the crack width, whenever the tolerable crack width has to be less than 0.30 mm, as is the case of aggressive environment:

$$w_k \text{ (mm)} = \beta s_{rm} \epsilon_{sm} \quad (\text{see Eq. 6})$$

The parameters in this expression are detailed in equations 7 and 8 of the text.

2. Prestressed Pretensioned Beams

(a) Steel reinforcement level:

$$s_{max} = 5.85 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (\text{see Eq. 14a})$$

(b) Tensile face of concrete:

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

3. Prestressed Post-Tensioned Beams

(a) Steel reinforcement level

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

(b) Tensile face of concrete

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

For non-bonded beams, the factor 6.51 become 6.83.

For high-strength high-performance prestressed concrete beams, with cylinder compressive strengths in the range of 10,000 –15,000 psi, the maximum crack width is reduced to

$$w_{max} \text{ (in.)} = 2.74 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \quad (\text{see Eq. 16a})$$

In SI units,

$$w_{\max} (mm) = 4.0 \times 10^{-5} \frac{A_s}{\sum o} (\Delta f_s) \quad (\text{see Eq. 16b})$$

4. Prestressed Concrete Circular Tanks

$$w_{\max} (in.) = 4.1 \times 10^{-6} \varepsilon_{ct} E_{ps} \sqrt{G_I} \quad (\text{see Eq. 19a})$$

5. Two-way Action Structural Slabs and Plates

$$w_{\max} = K \beta f_s \sqrt{G_1}, \quad \text{where } G_1 = \frac{s_1 s_2 d_c}{d_{b1}} \times \frac{8}{\pi} \quad (\text{see Eqs. 21,22})$$

6. Deep Beams and End-Anchorage blocks

For deep beams whose effective depth, d , exceeds 36 in. (915 mm), side face crack control is exercised by having skin reinforcement uniformly distributed on both vertical faces at a distance $d/2$ nearest the flexural tension reinforcement. The high concentrated prestressing forces at the end anchorage blocks cause splitting and bursting cracks that can result in the failure of such beams, particularly those that are post-tensioned. It is imperative to provide closely-spaced closed confining rectangular hoops at the end zones as required by analysis. AASHTO Specifications give detailed guidelines on the design of anchorage zone reinforcement to prevent bursting and spalling cracks, primarily using the strut-and tie approach for the proportioning of the reinforcement.

APPENDIX

Example 1 - Reinforced Concrete Beam in Severe Environment (Reproduced from the author's textbook, Ref. 2)

A reinforced concrete beam, 12 in. wide and a clear cover of 1.5 in. A-1 is part of a structure subjected to severe environment. Check its crack control performance by the

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following two methods given that $f'_c = 5.0$ ksi, $f_y = 75$ ksi, $A_s = 3$ No. 8 bars.

(a) Gergely-Lutz crack control expression for the maximum crack width, w , in beams and one-way slabs:

$$w = 0.076 \beta f_s \sqrt[3]{d_c A} \times 10^{-3} \text{ in.}$$

where $\beta = 1.20$, A = concrete area in tension/ number of bars, $f_s = 0.6 f_y$

(b) Reinforcement spacing approach by the ACI 318-99.

Solution:

(a) Equation approach

$$f_s = 0.6 f_y = 0.6 \times 75 = 45 \text{ ksi, } d_c = 2.5 \text{ in.}$$

$$A = \frac{(2 \times 2.5)12}{3} = 20 \text{ in.}^2, \text{ hence the crack width is}$$

$$w = 0.076 \times 1.2 \times 45 \sqrt[3]{2.5 \times 20} \times 10^{-3} = 0.0151 \text{ in. (0.38 mm).}$$

The allowable crack width according to Euro EC2, or CEB-FIP = 0.30 mm for normal environment and less for severe environment.

The ACI 224 tolerable width = 0.012 in. (0.30 mm). Hence, the 3 No. 8 bars are non-effective in crack control according to both ACI 224 and Euro EC2.

(b) Proposed ACI 318-99 Approach

$$\text{Max. allowable spacing } s = 540/f_s - 2.5 c_c = \frac{540}{45} - 2.5 \times 2 = 7 \text{ in. (179 mm)}$$

But not to exceed $12(36/f_y) = 12 \times 36 / 45 = 9.6 \text{ in. (244 mm)} > 7 \text{ in.}$, hence a maximum allowable spacing of 7 in. controls.

Actual spacing $s = \frac{1}{2} (12 - 2 \times 2.0 \text{ in. side cover} - 2 \times 0.5 \text{ in. for stirrups} - 2 \times 0.5 \text{ in. to center of outside bars})$; , or $s = 3 \text{ in. center to center, } < 7 \text{ in. allowable}$; hence the beam section is O.K. for crack control by the current ACI 318 provisions, although it fails when applying the tolerable crack width criteria of Table 1, and also fails by the Euro EC2 requirements. One solution would be to use a larger number of smaller diameter bars in two layers to reduce the value of "A", leading to smaller computed crack width.

Example 2 – Post-Tensioned Prestressed Concrete Flanged Section Containing

Non- Prestressing Reinforcement (Reproduced from the author's textbook in Ref. 11): A post-tensioned prestressed concrete T- beam is prestressed with twelve 7 / 16 -in. dia. 7-wire strands of 270-K grade and additionally reinforced with four #6 non-prestressed mild steel reinforcement. The geometry of the section is as shown in Figure 9. Given that $f'_c = 5\,000$ psi, $E_c = 57\,000\sqrt{f'_c}$ psi, and $E_s = 28\,000$ ksi, find the mean stabilized crack spacing and the crack widths at the reinforcement level, as well as at the tensile face of the beam at $\Delta f_s = 30\,000$ psi, assuming there is no failure in shear or bond. Then determine whether the beam satisfies the serviceability criteria for crack control for humidity and moist air.

Solution:

$$\Delta f_s = 30\,000 \text{ psi} = 30 \text{ ksi}$$

Mean Stabilized Crack Spacing:

$$A_t = 8 \times 12 = 96 \text{ in.}^2$$

$$\Sigma o = 12 \times \pi \times \frac{7}{16} + 4 \times 2.36 = 25.93 \text{ in.}$$

$$a_{cs} = 1.54 \frac{A_t}{\Sigma o} = 1.54 \times \frac{96}{25.93} = 5.70 \text{ in. (145 mm)}$$

Maximum Crack Width at Reinforcement Level:

$$w_{\max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) = 6.51 \times 10^{-5} \times \frac{96}{25.93} \times 30 = 0.0072 \text{ in. (0.18 mm)}$$

Maximum Crack Width at Tensile Face of Beam:

$$R_t = \frac{22 - 9.31}{22 - 9.31 - 4} = 1.46$$

$$w'_{\max} = w_{\max} R_i = 0.007 \times 1.46 = 0.0102 \text{ in. (0.26 mm)}$$

Maximum Tolerable Crack Width for Humidity:

From Table 1, the maximum tolerable crack width for the stated humidity conditions is 0.012 in. (0.3 mm) > 0.0102 in. (0.26 mm). O.K.

Example 3 - Crack-control Evaluation for Serviceability in a Rectangular Two-way Floor Panel Subjected to Severe Exposure Conditions. (Reproduced from the author's textbook, Ref. 2): Select the bar size and spacing necessary for crack control at the column reaction region of the 7-in.-thick slab that is uniformly loaded. The ratio of the short span to the long span is 0.8. Select the bar size for two conditions:

Condition A: Floor is subjected to severe exposure of humidity and moist air.

Condition B: Floor sustains an aggressive chemical environment where the design working stress level in the reinforcement is limited to 15 ksi (15,000 psi), given:

$$\beta = 1.20, \quad \frac{l_s}{l_l} = 0.8, \quad f_s = 60 \text{ ksi (414 MPa)}$$

Solution:

Condition A: Humidity and moist air

Tolerable $w_{\max} = 0.012 \text{ in. (0.3 mm)}$ (Table1). Try No. 4 bars $d_b = 0.5$, $d_c = 0.75 + 0.25 = 1.0 \text{ in.}$ Assume that $s_1 = s_2 = s$ for the given panel.

The aspect ratio, $l_y/l_l = 0.8$. $K = 2.1 \times 10^{-5}$ for concentrated reaction at the column support (Table4), to give $G_I = 394 \text{ in.}^2$. Therefore,

$$394 = \frac{s^2 d_c}{d_{bl}} \times \frac{8}{\Pi} = \frac{s^2 \times 1.0}{0.5} \times \frac{8}{\Pi} \quad 8.8 \text{ in.}$$

Hence use No. 4 bars at 8Qw in. center to center each way for crack control.

Condition B: Aggressive chemical environment

Tolerable $w_{\max} = 0.007$ in. (0.18 mm) (Table 1). $f_s = 15$ ksi to be used as a low stress level for sanitary or water-retaining structures instead of $0.4 f_y$.

Try No. 5 bars ($d_{b1} = 0.625$ in.), to give a grid index $G_I = 343$ in.²

$$d_{c1} = 0.75 + 0.312 = 1.06 \text{ in.}$$

$$G_I = 343 = \frac{s^2 \times 1.06}{0.625} \times \frac{8}{\Pi} \quad \text{to get } s = 8.9 \text{ in.}$$

Use No. 5 bars at 9-in. (229-mm) center-to-center spacing each way for crack control.

Reinforcement summary:

Condition A: No. 4 bars at 8 in. c–c (12.7-mm diameter at 216 mm c/c)

Condition B: No. 5 bars at 9 in. c–c (15.9-mm diameter at 229 mm c/c)

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METRIC (SI) UNIT EQUIVALENTS

1 inch	= 25.4 mm	1 lb	= 4.448 N
1 foot	= 0.305 m	1 kip	= 4448 N
1 in. ²	= 645.16 mm ²	1 lb/ft	= 14.594 N/m
1 in. ³	= 16.387 mm ³	1 kip/ft	= 14.594 kN/m
1 in. ⁴	= 416.231 mm ⁴	1 kip-in.	= 113 N-m
1 psi	= 6.895 Pa	$1\sqrt{f'_c} \text{ psi}$	= $0.083036\sqrt{f'_c} \text{ MPa}$

Table 1. Maximum tolerable flexural crack widths³

Exposure condition	Crack width	
	in.	mm
Dry air or protective membrane	0.016	0.41
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

Table 2. Maximum bar diameters for high-bond bars⁹

Steel Stress (MPa)	Maximum bar size (mm)
160	32
200	25
240	20
280	16
320	12
360	10
400	8
450	6

Table 3. Maximum bar spacing for high-bond bars⁹

Steel Stress (MPa)	Maximum Bar Spacing (mm)	
	Pure Flexure	Pure Tension
160	300	200
200	250	150
240	200	125
280	150	75
320	100	--
360	50	--

Table 4. Fracture coefficients for slabs and plates⁵

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, hinges.

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

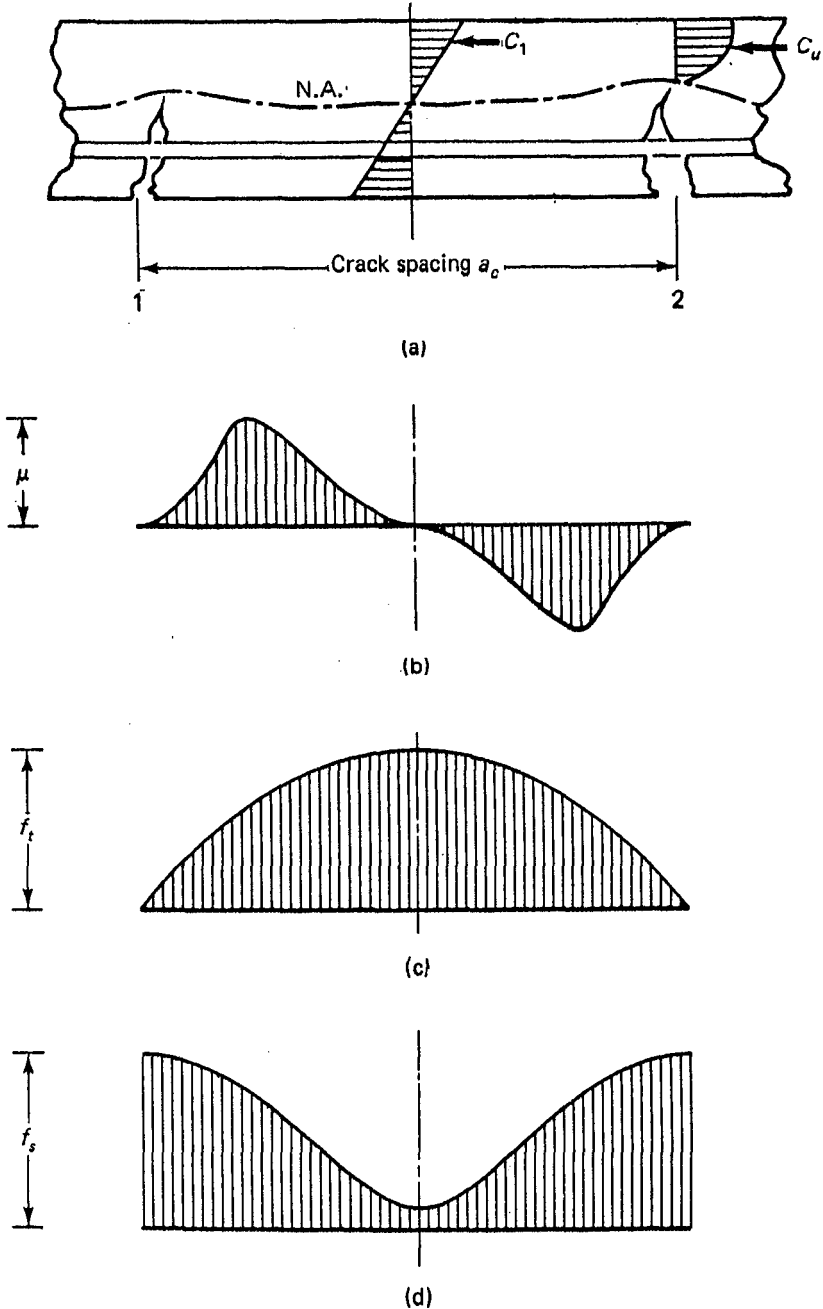


Figure 1. Schematic stress distribution between two flexural cracks:
(a) Section between two principal cracks; (b) Bond Stress distribution; (c) Longitudinal tensile stress in the concrete;
(d) Longitudinal tensile stress in the reinforcement.

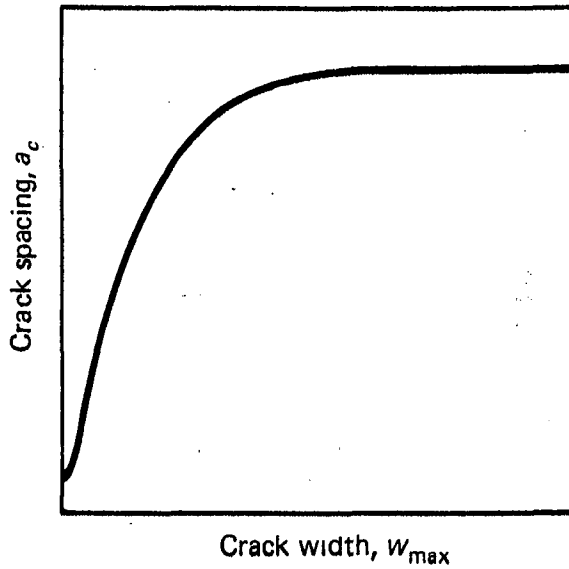


Figure 2. Schematic variation of crack width with crack spacing.

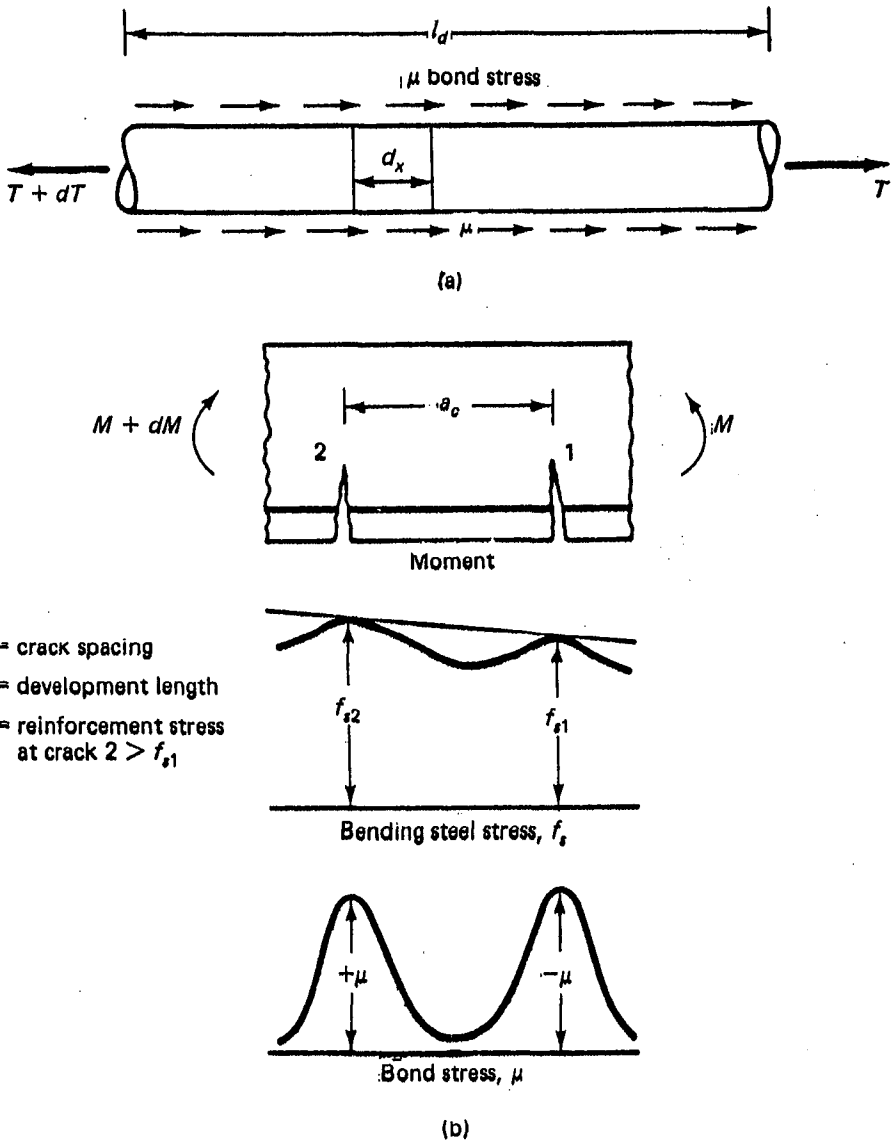
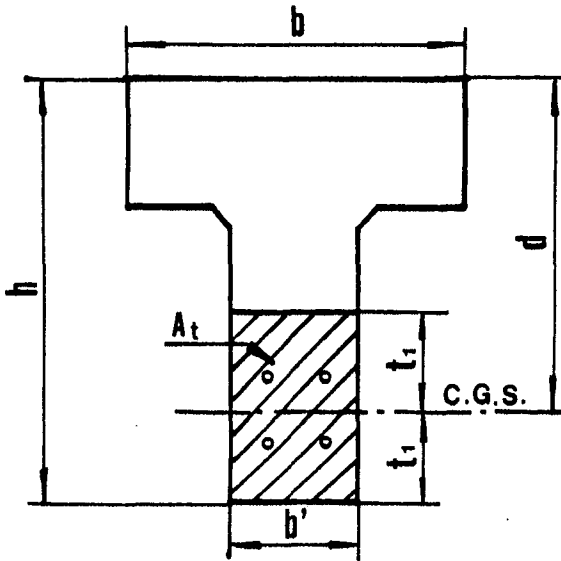
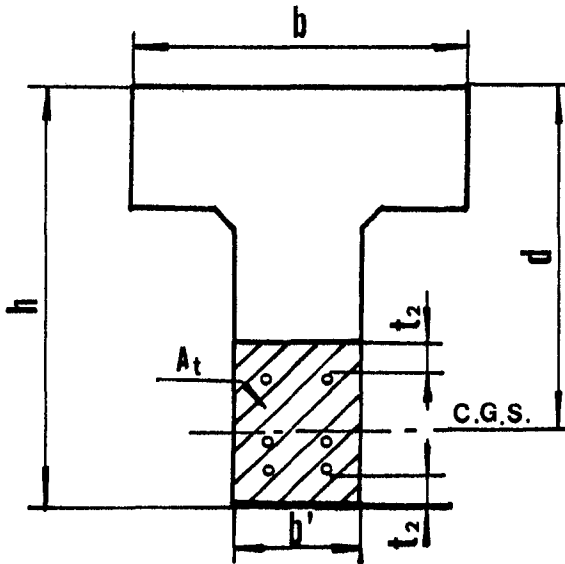


Figure 3. Forces and stress distribution in prestressed beam stabilized cracks: (a) longitudinal forces, (b) bending and bond stress distribution.



(a) For even distribution of reinforcement in concrete



(b) For non-even distribution of reinforcement in concrete

Figure 4. Effective concrete area in tension: (a) even reinforcement distribution, (b) non-even reinforcement distribution.

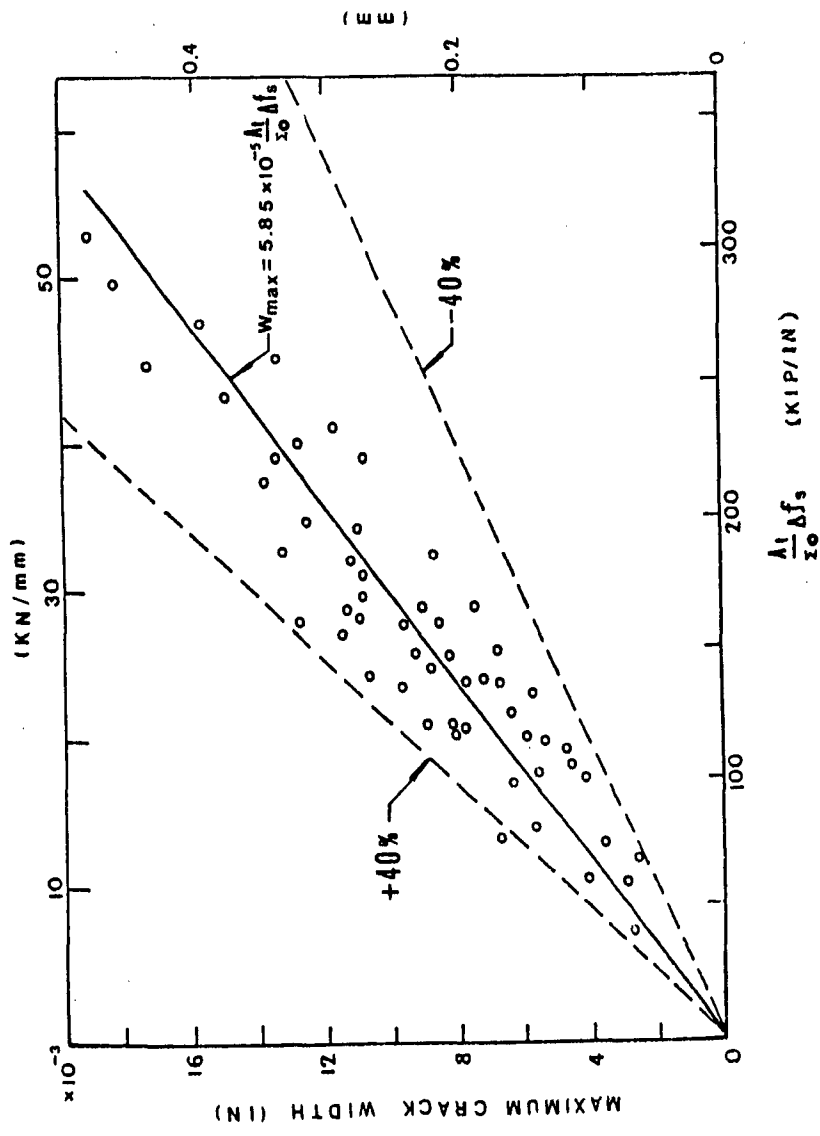


Figure 5. Linearized Maximum Crack Width vs. $\left(\frac{A_1}{\Sigma_o}\right) \Delta f_s$ In Pretension Beams

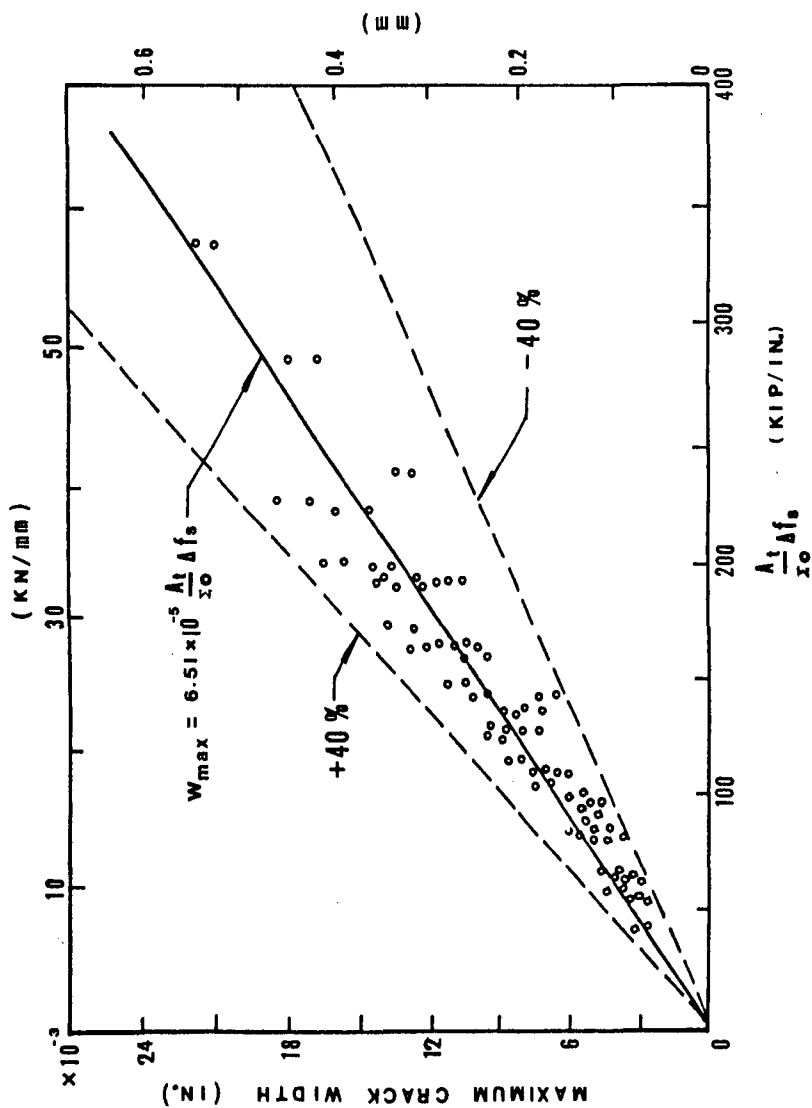


Figure 6. Linearized Maximum Crack Width vs. $\left(\frac{A_t}{\Sigma_o}\right) \Delta f_s$ In Post-tensioned Beams

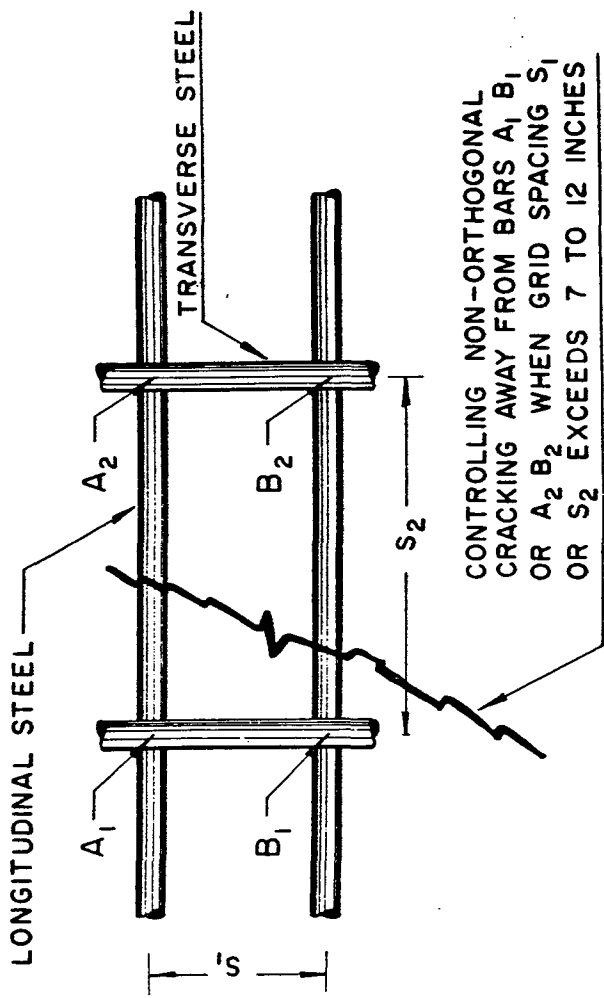


Figure 7. Grid unit in two-way action reinforcement.

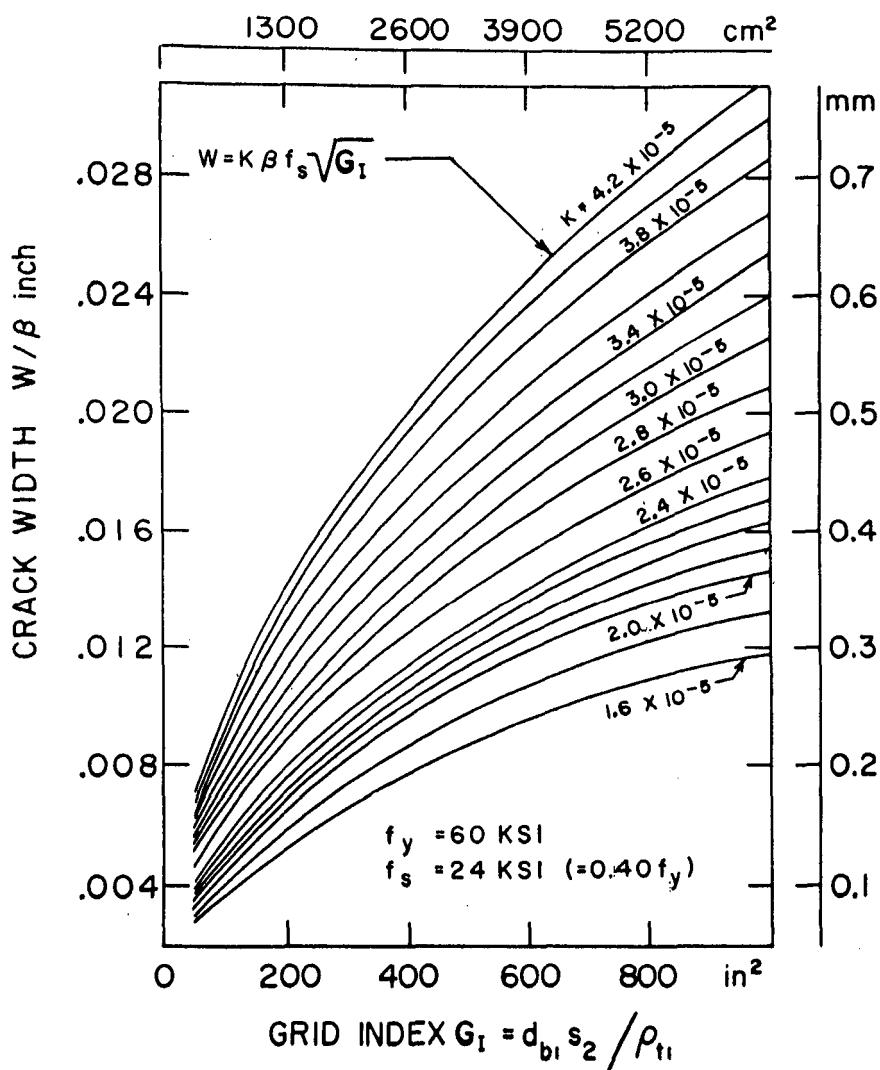


Figure 8. Crack control reinforcement distribution in two-way action slabs and plates for all exposure conditions: $f_y = 60 \text{ ksi}$, $f_s = 0.40$, $f_y = 24 \text{ ksi}$.

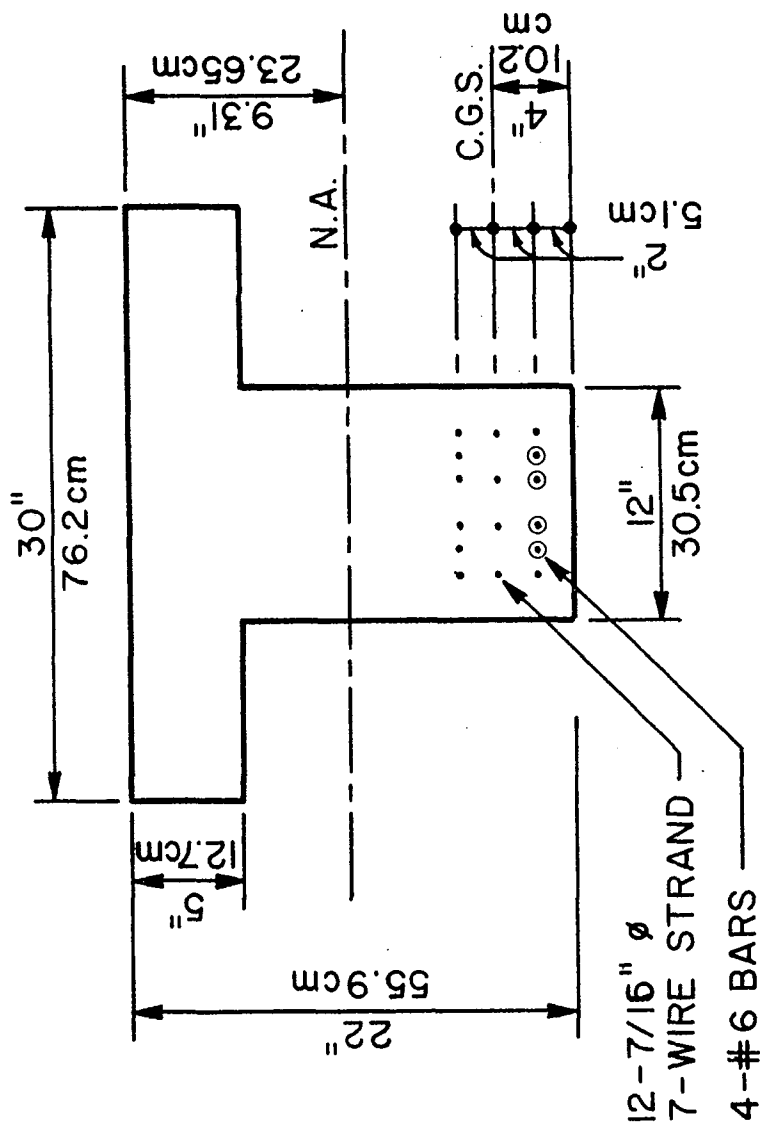


Figure 9. Geometrical dimensions of prestressed beam in example 2.

Early-Age Thermal Cracking in Laser-Screeded Concrete Slabs

by H. Haynes

Synopsis: A case history is presented in which a laser-screeded slab showed more cracks than would be expected in conventional strip-cast slabs. It was determined that laser-screeded slabs are more sensitive to early-age, thermal cracking than strip-cast slabs because of extra restraint provided by fixed-edge boundary conditions. Among the possible solutions is closer spacing of contraction joints and fog curing during the first day.

Keywords: case history; concrete slabs; cracks; curling; early age; fog curing; laser screed; slab-on-grade; temperature and shrinkage; thermal strain; thermal stress

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INTRODUCTION

Laser-guided self-propelled screeds (Figure 1) for slab-on-grade construction were introduced in 1986. One machine can screed 40,000 ft² (3,700 m²) in one day, about four times the production rate of conventional strip-cast construction. The cost saving of using laser-screed equipment is significant, and its use for construction of large slabs is already an industry standard.

New developments can bring new problems. This paper addresses a field problem with laser-screeded slabs. The author, a consulting concrete engineer in the San Francisco Bay area of California, has come across two projects where laser-screeded slabs showed more cracking than expected. The cracks seemed to be related to thermal contraction and drying shrinkage. Few details of these projects were known.

The author later had an opportunity to investigate a similar problem that is discussed in detail here. Evaluation of the findings suggests that laser-screeded slabs may be more sensitive to cracking due to early-age thermal behavior than conventional strip-cast slabs.

CASE HISTORY

A tilt-up warehouse located in Fremont, California, was built in 1998 having a slab-on-grade approximately 209,000 ft² (19,400 m²). Laser-screed equipment was used to place the slab during six operating days. The average area of slab cast was about 35,000 ft² (3200 m²) per day. In order to assure a continuous supply of concrete, placement operations were conducted from 1 AM to 8 AM.

The slab was 5 in. (127mm) thick and reinforced with #3 bars at 18 in. (460 mm) each way. Contraction joints spaced at 16-ft. (4.9 m) intervals in one direction and 20-ft. (6.1 m) intervals in the other were saw-cut to a depth of 1 in. (25 mm) immediately after the final finishing operation. The sub-base conditions were 1 in. (25 mm) of sand over 5 in. (127 mm) of Class II base rock. The concrete used Type II portland cement with a total cementitious material content of 526 lb/cy (310 kg/m³) of which 15% by mass was Type F fly ash. The water to cementitious material ratio was 0.53 and the specified slump was 3 in. (75 mm). A water-reducing admixture was used. A quartzitic sand was used as fine aggregate, and limestone 3/4-in. (9 mm) nominal maximum size as coarse aggregate. The compressive strength of the concrete at 28 days averaged 4320 psi. (29.8 Mpa)

The slab showed cracks between contraction joints (some with multiple cracks) within days of placing the concrete. After six months of fork-lift traffic, cracks in the aisles about 0.030-in. (0.8 mm) and wider showed raveling along the edges. Vertical movement of the slab could be felt as a fork-lift drove across one of the cracks. Additional cracks appeared in the traffic aisles.

The following common causes of cracking were ruled out:

5. Core samples from the slab showed that the thickness was uniformly 5 in. (127 mm) and the compressive strength was 5,060 psi. (34.9 Mpa)
6. The depth of saw cut contraction joints measured 1 in. (25mm)
7. During placement of concrete, the slump was consistently between 4 to 4.5 in. (100 to 114 mm) The increase in slump from that of the proposed mixture was 10%