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R. F. Warner

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PRESTRESSED CONCRETE BRIDGE MEMBERS

PROGRESS REPORT 20

THE CALCULATION OF FLEXURAL STRESSES
IN A PRESTRESSED CONCRETE MEMBER

by

Robert F. Warner

FRITZ ENGINEERING LABORATORY
LEHIGH UNIVERSITY
BETHLEHEM, PA.

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ERRATA:

In Pages 3 and 12

\[ e_{cs} = \text{the tensile strain which occurs in the concrete at the steel level during the application of the moment } M, \text{ minus the value of } e_{ce}. \]
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The research program on prestressed concrete bridge members is under the direct supervision of Professor Carl E. Ekberg, Jr.; Professor W. J. Eney is Director of the Fritz Engineering Laboratory and Head of the Department of Civil Engineering.
SYNOPSIS

The necessity of knowing the flexural stresses induced in a prestressed member by an applied moment arises in a number of instances, notably when the flexural fatigue properties are to be determined.

Equations are here set up for the calculation of the stress-moment relations for the steel reinforcement and the extreme fibers of the concrete of a rectangular, prestressed concrete section. The equations indicate the state of stress in the section as the applied moment is increased from zero to the failure point.
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INTRODUCTION

1. Introduction

Before the flexural fatigue properties of a prestressed concrete member can be calculated it is necessary to know, for a number of cross sections in the member, the relation between applied moment and the resulting stresses in the steel reinforcement and in the extreme fibers of the concrete. In this report equations are set up for the calculation of the stress-moment relations for a rectangular, prestressed concrete section.

Depending upon the magnitude of the applied moment, there may or may not be flexural cracks present in the concrete. In either case the steel and concrete stresses are calculated using the following data:

(I) Equations of static equilibrium
(II) The assumption of a linear strain distribution in the cross sections of the concrete.
(III) The given stress-strain relation for the steel
(IV) An assumed stress-strain relation for the concrete

Prior to cracking the concrete and steel stresses are reasonably small and a linear relation between stress and strain
may be assumed for both materials. Thus the stresses in an uncracked section are calculated in a reasonably straightforward manner.

At higher stages of loading the concrete stress-strain relation is noticeably non-elastic and a more complicated relation must be assumed. The complexity of the problem is further increased by the possibility of yielding in the steel and by the fact that, after cracking, the position of the neutral axis becomes an unknown quantity to be determined. The solution for the cracked section is therefore more lengthy than for the uncracked case. However a direct solution has been made possible by the construction of an intercept chart which removes the necessity of solving simultaneous equations.
2. Notation and Sign Convention

The following notation, which follows that suggested by the A. C. I. - A.S.C.E. Joint Committee 323*, is used.

\[ \alpha = \frac{E_c}{f'_c} \cdot e_{cu} \]

- \( a \) = internal lever arm
- \( A_c = b \cdot h \) = area of entire concrete section
- \( b \) = width of beam
- \( C \) = total compressive force in the concrete
- \( d \) = depth to C.G. of steel
- \( e \) = eccentricity of C.G. of steel with respect to the C.G. of the concrete area
- \( e_c \) = concrete strain
- \( e_{cs} \) = total concrete strain at the steel level
- \( e_{cu} \) = ultimate concrete strain
- \( e_{cl} \) = concrete strain at the top fibre
- \( e_s \) = steel strain
- \( E = \frac{e_c}{e_{cu}} \)
- \( E_l = \frac{e_{cl}}{e_{cu}} \)

* "Proposed Definitions and Notations for Restressed Concrete"

\[ E_C = \text{initial modulus of elasticity of the concrete} \]

\[ f_c = \text{concrete stress} \]

\[ f'_c = \text{concrete ultimate stress} \]

\[ f_s = \text{steel stress} \]

\[ f_{se} = \text{steel stress due to effective prestress} \]

\[ F = \frac{f_c}{f'_c} \]

\[ F = \text{effective prestress force after deduction of all losses} \]

\[ f^t_L = \text{concrete stress in top fiber due to applied moment } M_L \]

\[ f^b_L = \text{concrete stress in bottom fiber due to applied moment } M_L \]

\[ f^t_F = \text{concrete stress in top fiber due to the effective prestressing force } F \]

\[ f^b_F = \text{concrete stress in the bottom fiber due to the effective prestressing force } F \]

\[ h = \text{total depth of beam} \]

\[ I = \text{total moment of inertia of the uncracked section} \]

\[ I_C = \text{moment of inertia of the uncracked concrete} \]

\[ k_d = \text{depth to the neutral axis of the cracked section} \]

\[ k_2 k_d = \text{depth to the resultant compressive force in the cracked section} \]

\[ k_2' = (1-k_2) \]

\[ n = \frac{E_s}{E_c} \]
\[ p = \frac{A_s}{b_d} \]
\[ p' = \frac{A_s}{b_h} \]

\( T = \text{total tensile force at a cracked section} \)

\( \bar{x} = \text{distance from C.G. of the steel to the N.A. of the composite uncracked section.} \)

Sign Convention: In the following, compressive stresses are taken as positive and tensile stresses as negative.
3. Uncracked Section, Approximate Solution

In the loading stage prior to cracking, a first approximation to the behavior of a prestressed member may be obtained by neglecting the increase in steel stress, which occurs with increased moment as a result of the small elastic deformation in the beam. Then we may write:

\[ M = F \cdot a \]

where

- \( M \) = applied moment
- \( F = f_{se}A_s \) = effective prestressing force in the steel
- \( a \) = lever arm, i.e., the distance between the center of gravity of the compressive force and the center of gravity of the tension force.

It follows that "a" increases linearly with \( M \) as shown in Figure I.

The concrete stresses are obtained as the sum of the stresses at initial prestress plus the stresses due to the applied load. Thus, with compressive stresses positive,

\[ f_c^t = f_F^t + f_L^t \]

\[ f_c^b = f_F^b + f_L^b \]

but

\[ f_F^t = \left( \frac{F}{A_c} - \frac{F \cdot e}{I_c} \cdot \frac{h}{2} \right) \]
i.e.\[ f_T^c = + \frac{F}{A_c} (1 - \frac{6e}{h}) \]

and\[ f_L^c = + \frac{M}{I_c} y \]

and so\[ f_c^b = \frac{F}{A_c} (1 + \frac{6e}{h}) - \frac{6M}{bh^2} \]

similarly\[ f_c^b = \frac{F}{A_c} (1 + \frac{6e}{h}) - \frac{6M}{bh^2} \]

The steel stresses have of course been assumed to remain constant at \( f_{se} \), the steel stress due to effective prestress.

The error involved in the above method may be expected to be less than 10%.

4. Stresses at an Uncracked Section

To take into account the variation in steel stress, the analysis is made by treating the section as a composite member, calculating the stresses resulting from the applied moment, M, and adding them, as before, to the initial stresses due to the effective prestress.

Referring to Figure 2, the position of the neutral axis of the composite section is found as:

\[ \bar{x} = \frac{A_c \cdot e}{A_c + (n-1)A_s} \]
where \( e \) is the distance from the center of gravity of the concrete area \( A_c \) to the center of gravity of the steel area \( A_s \), and \( \bar{x} \) is the distance from the center of gravity of the steel area to the neutral axis.

The moment of inertia of the composite section is

\[
I = \frac{b \cdot h^3}{12} + b \cdot h (e - \bar{x})^2 + (n-1)A_s \bar{x}^2
\]

i.e.

\[
I = A_c \left[ \frac{h^2}{12} + (e - \bar{x})^2 + (n-1)p' \cdot \bar{x}^2 \right]
\]

where \( p' = \frac{A_s}{b \cdot h} \) = Proportion of steel reinforcement to total concrete area.

The stresses in the concrete due to the applied moment \( M_L \) are:

\[
f^c_L = + \frac{M}{I} \left( \frac{h}{2} + e - \bar{x} \right)
\]

\[
f^b_L = - \frac{M}{I} \left( \frac{h}{2} - e + \bar{x} \right)
\]

and the total concrete stresses are given by

\[
f_c = f_F + f_L
\]
i.e.

\[ f_C^c = + \frac{F}{A_c} \left( 1 - \frac{6e}{h} \right) + \frac{M}{I} \left( \frac{h}{2} + e - \bar{x} \right) \]  

\[ f_C^b = + \frac{F}{A_c} \left( 1 + \frac{6e}{h} \right) - \frac{M}{I} \left( \frac{h}{2} - e + \bar{x} \right) \] 

The additional steel stress due to \( M \) is

\[ f_{SL} = n \frac{M}{I} \bar{x} \]

and the total steel stress is therefore

\[ f_s = f_{se} + n \frac{M}{I} \bar{x} \]

or

\[ f_s = \frac{F}{A_s} + n \frac{M}{I} \bar{x} \]

It should be noted that in the above equations "\( I \)" refers to the total moment of inertia of the cross-section and is given by Equation 7.
5. Stress-Strain Relation for Concrete

A cubic parabola is here used to represent the stress-strain relation for the concrete and is plotted, in dimensionless form, in Figure 3. The general equation for such a curve,

\[ F = A_1 E^3 + A_2 E^2 + A_3 E + A_4 \]

where \( F = \frac{f_c}{f'} \)

and \( E = \frac{\varepsilon_c}{\varepsilon_{cu}}, \)

is convenient to work with, but at the same time contains a sufficient number of coefficients, \( A_1, A_2, A_3, A_4 \), to give a reasonable approximation to the actual stress-strain relation.

The coefficients are evaluated such that

(a) when \( E = 0, \frac{dF}{dE} = \alpha = \frac{E_c}{f'} \cdot \varepsilon_{cu} \) (i.e. the slope of the stress-strain curve at zero stress is equal to the initial modulus of elasticity of the concrete)

(b) when \( E = 1.0, \frac{dF}{dE} = 0 \) (the slope of the stress-strain curve is zero at ultimate stress)

(c) when \( E = 0, F = 0, \) and

(d) \( E = 1.0, F = 1.0 \)
The resulting expression, in dimensionless form, is

\[ F = \alpha \cdot E - (2\alpha - 3)E^2 + (\alpha - 2)E^3 \]

where \( \alpha = \frac{E_c}{f_c} \cdot \epsilon_{cu} \)

or, in the more usual notation,

\[ \frac{f_c}{f_c^*} = \frac{E_c}{f_c^*} \cdot \epsilon \cdot \left( \frac{e_c}{\epsilon_{cu}} \right) - \left[ \frac{2E_c}{f_c^*} \cdot \epsilon - 3 \right] \left( \frac{e_c}{\epsilon_{cu}} \right)^2 + \left[ \frac{E_c}{f_c^*} \cdot \epsilon - 2 \right] \left( \frac{e_c}{\epsilon_{cu}} \right)^3 \]

For equation 11 to represent a monotonically increasing curve between \( E = 0 \) and \( E = 1 \), as shown in Figure 3, a limitation must be placed on the value of \( \alpha \), the initial slope of the curve. If the initial slope is too steep, the curve reaches its maximum value at a smaller value of \( E \) and then becomes a minimum at \( E = 1 \). This is illustrated in Figure 3.

Thus it is stipulated that

\[ \frac{d^2F}{dE^2} \leq 0 \text{ at } E = 1.0 \]

i.e.

\[ -2(2\alpha - 3) + 3 \times 2 \times (\alpha - 2)E \leq 0 \text{ at } E = 1.0 \]

\[ \therefore \alpha \leq 3 \]

The desirable values of \( \alpha \) to give a good approximation to the actual stress-strain relation are discussed later.
6. Strain Distribution

A linear strain distribution is assumed at the section under consideration, as in Figure 4. The total strain in the steel, when moment M is applied, is

\[ e_s = e_{se} + e_{ce} + e_{cs} \]

where

- \( e_{se} \) = strain in the steel due to effective prestress
- \( e_{ce} \) = strain in the concrete at the steel level due to effective prestress.
- \( e_{cs} \) = total strain in concrete at steel level after the moment M is applied,

also

- \( e_{cl} \) = total strain in concrete at the top fiber after the moment M is applied

The usual assumption of "perfect" bonding is implied in the above equation.

Thus the total steel strain, \( e_s \), can be thought of as the sum of two components, \( e_{cs} \) and \( (e_{se} + e_{ce}) \). The latter quantity may be considered to have a constant value for a given beam. For a pretensioned beam, it is equal to the total initial steel pre-stress \( e_{si} \) minus inelastic losses. If the losses can be ignored

\[ (e_{se} + e_{ce}) = e_{si} \]
Rewriting equation 14

\[ e_s - (e_{se} + e_{ce}) = e_{cs} \] \hspace{1cm} 15

From Figure 5, it can be seen that

\[ \frac{e_{cl}}{kd} = \frac{e_{cl} + e_{cs}}{d} \]

i.e.

\[ e_{cs} = \frac{1-k}{k} e_{cl} \] \hspace{1cm} 16a

dividing throughout by \( e_{cu} \) we obtain

\[ \frac{e_{cs}}{e_{cu}} = \frac{1-k}{k} \frac{e_{cl}}{e_{cu}} \] \hspace{1cm} 16

Now the strain at a distance "x" above the level of the neutral axis is

\[ e_c = e_{cl} \cdot \frac{x}{kd} \] \hspace{1cm} 17a

and again dividing by \( e_{cu} \)

\[ \frac{e_c}{e_{cu}} = \frac{e_{cl}}{e_{cu}} \cdot \frac{x}{kd} \] \hspace{1cm} 17b
or
\[ E = \frac{\varepsilon_{cl}}{\varepsilon_{cu}} \frac{x}{kd} \] .......................... 17

hence
\[ \frac{dE}{dx} = \frac{\varepsilon_{cl}}{\varepsilon_{cu}} \frac{1}{kd} \] .......................... 18

Equations 16 and 18 will be used later.

7. Equilibrium Equations

Considering the concrete compressive stress block above the neutral axis of the cracked section, as shown in Figure 5, we have:

\[ C = b \int_0^{kd} f_c \, dx \] .......................... 19

which may be written as

\[ C = b \cdot f_c \int_0^{\varepsilon_{cl}} \frac{f_c}{f'_c} \frac{dx}{d\varepsilon} \, d\varepsilon \]

or, with \( \frac{f_c}{f'_c} = F \),
\[
\frac{e_c}{e_{cu}} = E
\]

and
\[
\frac{e_{c1}}{e_{cu}} = E_1,
\]

\[
C = b \cdot f_c \int_0^{E_1} F \frac{dx}{dE} \text{ dE}
\]

but from equation 18
\[
\frac{dx}{dE} = \frac{kd}{E_1}
\]

hence we can rewrite (20) in dimensionless form as

\[
\frac{C}{b \cdot d \cdot f_c} = \frac{1}{E_1} \cdot k \int_0^{E_1} F \text{ dE}
\]

i.e.
\[
\frac{C}{b \cdot d \cdot f_c} = \frac{k}{E_1} \left[ \frac{\alpha}{2} E_1^2 - \frac{2\alpha-3}{3} E_1^3 + \frac{\alpha-2}{4} E_1^4 \right]
\]

or
\[
\frac{C}{b \cdot d \cdot f_c} = k \left[ \frac{\alpha}{2} E_1 - \frac{2\alpha-3}{3} E_1^2 + \frac{\alpha-2}{4} E_1^3 \right] \ldots 22
\]
In the usual notation this is

\[ \frac{C}{b.d.f} = k \left\{ \frac{1}{2} \cdot \frac{E_c}{f_c} \cdot e_{cu} \left( \frac{e_{c1}}{e_{cu}} \right) + \left[ \frac{2}{3} \cdot \frac{E_c}{f_c} \cdot e_{cu} \cdot \frac{1}{e_{cu}} - 1 \right] \left( \frac{e_{c1}}{e_{cu}} \right)^2 \right\} + \left[ \frac{1}{4} \cdot \frac{E_c}{f_c} \cdot e_{cu} - \frac{1}{2} \right] \left( \frac{e_{c1}}{e_{cu}} \right)^3 \] ...

8. Value of \( \alpha \)

It is generally accepted that \( e_{cu} \), the ultimate strain in the top fiber of the beam, is a constant, the values quoted by different investigators lie between 0.003 and 0.004. It is also usual for the ratio \( \frac{E_c}{f_c} \) to be assumed, for practical purposes, a constant. Choosing \( e_{cu} \) as 0.003 and a value of 1000 for \( \frac{E_c}{f_c} \) the parameter \( \alpha \) becomes 3 which, as can be seen from equation 13, is the maximum value it may take.

The value \( \alpha = 3 \) ...

will be used here. Equation 22 then becomes

\[ \frac{C}{b.d.f} = k \left[ \frac{1}{4} E_1^3 - E_1^2 + \frac{12}{8} E_1 \right] \] ...
9. Position of the Center of Gravity of the Compressive Stress Block

To determine the value of the applied moment, $M$, in terms of the internal stresses, it is necessary to know the length of the internal lever arm and hence of the position of the resultant compressive force $C$. $M$ is given by

$$M = C d(1-k_2^2)$$

The parameter $k_2$ is chosen to represent the position of the force $C$ and may be found by a consideration of the geometry of the cubic parabola. Referring to Figure 6 it can be seen that the value of $k_2$ will depend upon the value of $E_1$, but although $E_1$ may vary between zero and unity only a small variation can occur in $k_2$. Limits to the value of $k_2$ can easily be found by replacing the parabola $OPA$, on the one hand by the straight line $OBA$, and, on the other, by the rectangle $OF_1A$; thus it is shown that the value of $k_2$ must always be between 0.5 and 0.33.

In evaluating $k_2$ it will be convenient to work with the quantity

$$k_2' = (1-k_2),$$
then, from Figure 7,

$$k_2^1 E_1 \int_0^E F \, dE = \int_0^E E \cdot F \cdot dE$$

considering the left hand side of the equation,

$$k_2^1 E_1 \int_0^E F \, dE = k_2^1 E_1 \int_0^E \left[ \alpha E - (2\alpha - 3)E^2 + (\alpha - 2)E^3 \right] dE$$

$$= k_2^1 \cdot \left[ \frac{\alpha}{2} \frac{E_1^3}{3} - (2\alpha - 3) \frac{E_1^4}{4} + (\alpha - 2) \frac{E_1^5}{5} \right]$$

and now considering the right hand side,

$$\int_0^E E \cdot F \cdot dE = \int_0^E \left[ \alpha E^2 - (2\alpha - 3)E^3 + (\alpha - 2)E^4 \right] dE$$

$$= \frac{\alpha}{3} E_1^3 - \frac{2\alpha - 3}{4} E_1^4 + \frac{\alpha - 2}{5} E_1^5.$$ 

Hence, with $\alpha = 3$,

$$k_2^1 = \frac{1 - \frac{3}{4} E_1 + \frac{1}{5} E_1^2}{\frac{3}{2} - E_1 + \frac{1}{4} E_1^2}$$
and
\[ k_2 = 1 - \frac{1 - \frac{3}{4} E_1 + \frac{1}{5} E_1^2}{\frac{3}{2} - E_1 + \frac{1}{4} E_1^2} \]  \hspace{1cm} 25

Equation 25 is plotted in Figure 7 which shows the variation to be from 0.33 for \( E_1 = 0 \), to 0.4 for \( E_1 = 1.0 \).

10. Calculation of Stresses

Summarizing the equations applying to the cracked section;

\[ F = aE - (2a-3)E^2 + (a-2)E^3 \]  \hspace{1cm} 11

\[ e_{cs} = e_s - (e_{se} + e_{ce}) \]  \hspace{1cm} 15

\[ \frac{e_{cs}}{e_{cu}} = \frac{1-k}{k} E_1 \]  \hspace{1cm} 16

\[ \frac{C}{b.d.f_c} = k \left[ \frac{1}{4} E_1^3 - E_1^2 + \frac{12}{8} E_1 \right] \]  \hspace{1cm} 24

\[ M = C.d (1-k_2k) = f_g A_g d (1-k_2k) \]  \hspace{1cm} 25

\[ k_2 = 1 - \frac{1 - \frac{3}{4} E_1 + \frac{1}{5} E_1^2}{\frac{3}{2} - E_1 + \frac{1}{4} E_1^2} \]  \hspace{1cm} 26
Equations (16) and (24), which contain as unknowns \( \frac{e_{cs}}{e_{cu}}, k, E_1 \) and \( \frac{C}{b.d.f_c^r} \), have been used to construct the intercept chart in Figure 7.

In obtaining the separate points on the stress moment curve, it will be convenient to first assume a steel stress, calculate the corresponding values of \( k, E, f_c \) and then the applied moment.

The procedure is as follows:

(a) choose values of \( e_s \) and \( f_s \) from a point on the stress-strain curve of the steel and find \( e_{cs} \) as

\[
e_{cs} = e_s = (e_{se} + e_{ce})
\]

and thence \( \frac{e_{cs}}{e_{cu}} \), remembering that \( (e_{se} + e_{ce}) \) is a known quantity for a given beam which depends only on the initial prestressing force and the losses. \( e_{cu} = 0.003 \).

(b) Calculate \( C = T = f_s A_s \) and hence \( \frac{C}{b.d.f_c^r} \)
(c) Enter the intercept chart shown in Figure 7 using the values of \( \frac{\varepsilon_{cs}}{\varepsilon_{cu}} \) and \( \frac{C}{b.d.f_c} \) given in steps (a) and (b), hence find \( k \) and \( E_1 \).

(d) Using the concrete stress strain relation Equation 11 and the value of \( E_1 \) from step (c), find \( F_1 \) and \( f_{c1} \).

(e) Calculate the moment \( M \) as

\[
M = f_s A_s d(1-k_2k)
\]

where \( k_2 \) can be read off Figure 7 for the corresponding value of \( E_1 \).

Thus the steel and concrete stresses \( f_s \) and \( f_{c1} \), given in steps (a) and (d), correspond to the moment \( M \) given in step (e).

11. Ultimate Strength

The ultimate flexural moment for the section is obtained from the equations in section 10 by placing \( F = 1.0 \) and \( E = 1.0 \).

\[
e_s = e_{se} + e_{ce} + e_{cu} \cdot \frac{1-k}{k} \]

... 27
The calculation is carried out by choosing a trial value of $f_s$, calculating $k$ with equation 28 and hence $e_s$ with equation 27. The correct value of $f_s$ has been chosen when the value of $e_s$, obtained from equation 27, agrees with the value obtained from the steel stress strain curve.
SAMPLE CALCULATION

12. Calculation of the Stress-Moment Relations for a Prestressed Beam

To illustrate the use of the equations, the stress-moment relations are now obtained for a prestressed beam. The details of the section, which are given below, refer to test beam A8 in the series of tests described in Progress Report 18. A full description of the manufacture and testing of the beam is given in that report.

The relevant data are,

\[ b = 8 \text{ in} \]
\[ h = 18 \text{ in} \]
\[ d = 13 \text{ in} \]
\[ A_s = 0.653 \text{ in}^2 \]
\[ f'_c = 6260 \text{ lb/sq. in} \]
\[ F_i = 96.33 \text{ kips}, f_{si} = \frac{96.33}{0.653} = 147.5 \text{ kips/in.}^2 \]
\[ F_0 = 92.47 \text{ kips}, f_{so} = \frac{92.47}{0.653} = 141.3 \text{ kips/in.}^2 \]
\[ F = 85.73 \text{ kips}, f_{se} = \frac{85.73}{0.653} = 131.2 \text{ kips/in.}^2 \]
\[ e_{se} + e_{ce} = 0.0060 \]
Ultimate Flexural Moment

Try \( f_S = 250 \text{kips/sq. in.} \quad e_S = 0.0127 \) (Figure 8)
\[ k = 0.335 \] (Equation 28)
\[ e_S = 0.0126 \] (Equation 27)

This is sufficiently close to 0.0127

\[ M = 250,000 \times 0.653 \times 13 \times (1 - 0.4 \times 0.335) \] (Equation 29)
\[ = 1.835 \times 10^6 \text{ in. lb.} \]

(Observed ultimate moment = \( 1.810 \times 10^6 \text{ in. lb.} \))

Uncracked Section

\[ \bar{x} = \frac{144 \times 4}{144 + (5-1)0.653} = 3.93 \text{ in} \] (Equation 6)

\[ I = \frac{8 \times 18^3}{12} + 8 \times 18 (4-3.93)^2 + (5-1)0.653 \times 3.93^2 \]
(Equation 7)

\[ I = 3940 \text{ in}^4 \]

(a) Zero Moment

\[ F = 85.73 \text{kips}, f_{se} = 131.2 \text{kips/sq in.} \]

\[ f_c = + \frac{85.73}{144} \left(1 - \frac{6 \times 4}{18}\right) \] (Equation 8)
\[ = -199 \text{ lb/sq in} \] (negative sign means tension)
\[ f^b_c = \frac{F}{A_c} \left(1 + \frac{6e}{h}\right) \quad \text{(Equation 8)} \]

\[ = + 1390 \text{ lb/sq in} \]

**Cracking Moment**

Taking \( f^\prime_t = -\frac{1}{10} f^b_c \)

\[ = - 626 \text{ lb/sq in} \]

and substituting values in equation 8

\[-626 = \frac{85.730}{144} (1 + \frac{24}{18}) - \frac{M}{3938} (9 - 4 + 3.93)\]

\[ M = 879,000 \text{ in lb} \]

(Observed cracking moment = 863,000 in lb)

\[ f_s = \frac{F}{A_s} + n \cdot \frac{M}{I} x \quad \text{(Equation 9)} \]

\[ = 137,000 \text{ lb/sq in.} \]

\[ f^\prime_c = -199 + \frac{879,000}{3938} (9 + 4 - 3.93) \quad \text{(Equation 8)} \]

\[ = 1840 \text{ lb/sq. in.} \]

**Cracked Section**

The calculations for the cracked section have been carried out in Tabular form on the following page. The procedure used is described on pages 20 and 21.
### Calculations for Cracked Section

<table>
<thead>
<tr>
<th>$f_s$ kips/in.</th>
<th>$\frac{f_s}{f_u}$</th>
<th>$f_s A_s$</th>
<th>$e_s$</th>
<th>$\frac{\Delta e_{cs}}{\Delta e_{cu}}$</th>
<th>$\frac{C}{b.d.f_c}$</th>
<th>$E_1$</th>
<th>$k$</th>
<th>$k_2$</th>
<th>$\frac{f_c}{f_c}$</th>
<th>$M$ kip-in.</th>
<th>$\frac{M}{M_u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>0.604</td>
<td>104.5</td>
<td>0.0064</td>
<td>0.0004</td>
<td>0.133</td>
<td>0.18</td>
<td>0.70</td>
<td>0.34</td>
<td>0.45</td>
<td>1 035</td>
<td>0.563</td>
</tr>
<tr>
<td>180</td>
<td>0.680</td>
<td>117.5</td>
<td>0.0069</td>
<td>0.0009</td>
<td>0.30</td>
<td>0.30</td>
<td>0.50</td>
<td>0.35</td>
<td>0.66</td>
<td>1 260</td>
<td>0.685</td>
</tr>
<tr>
<td>200</td>
<td>0.756</td>
<td>130.6</td>
<td>0.0076</td>
<td>0.0016</td>
<td>0.516</td>
<td>0.41</td>
<td>0.44</td>
<td>0.36</td>
<td>0.80</td>
<td>1 430</td>
<td>0.777</td>
</tr>
<tr>
<td>210</td>
<td>0.794</td>
<td>137.1</td>
<td>0.0080</td>
<td>0.0020</td>
<td>0.666</td>
<td>0.48</td>
<td>0.42</td>
<td>0.36</td>
<td>0.86</td>
<td>1 512</td>
<td>0.822</td>
</tr>
<tr>
<td>220</td>
<td>0.832</td>
<td>143.7</td>
<td>0.0085</td>
<td>0.0025</td>
<td>0.833</td>
<td>0.54</td>
<td>0.39</td>
<td>0.37</td>
<td>0.90</td>
<td>1 598</td>
<td>0.868</td>
</tr>
<tr>
<td>230</td>
<td>0.869</td>
<td>150.2</td>
<td>0.0093</td>
<td>0.0033</td>
<td>1.083</td>
<td>0.231</td>
<td>0.64</td>
<td>0.37</td>
<td>0.95</td>
<td>1 682</td>
<td>0.914</td>
</tr>
<tr>
<td>240</td>
<td>0.905</td>
<td>156.7</td>
<td>0.0101</td>
<td>0.0041</td>
<td>1.377</td>
<td>0.241</td>
<td>0.74</td>
<td>0.36</td>
<td>0.98</td>
<td>1 750</td>
<td>0.950</td>
</tr>
</tbody>
</table>
(a) Stresses due to effective prestressing force, \( F \).

(b) Stresses when moment \( M_L \), less than the cracking moment, is applied.

Figure 1: UNCRACKED SECTION

(Approximate Solution for Concrete Stresses)

Figure 2: UNCRACKED SECTION
Figure 3: CUBIC PARABOLA
Figure 4: CRACKED SECTION

Strain Distribution at Various Stages of Loading
Figure 5: CRACKED SECTION

Distribution of stress and strain above the Neutral Axis

Figure 6: CENTER OF GRAVITY OF STRESS BLOCK
Figure 7: INTERCEPT CHART
Figure 8: STRESS-STRAIN CURVE FOR 7/16" DIAMETER HIGH TENSILE STEEL STRAND
Figure 9: STRESS MOMENT RELATIONS

\( f_s = \text{steel stress} \)

\( f^b_c = \text{stress in concrete bottom fiber} \)

\( f^t_c = \text{stress in concrete top fiber} \)