1-1-1978

Prestress loss of pre-post-tensioned concrete bridge members.

Burt Hoffman

PRESTRESS LOSS OF PRE-POST-TENSIONED

CONCRETE BRIDGE MEMBERS

by

Burt Hoffman

A THESIS

Presented to the Graduate Committee

of Lehigh University

in Candidacy for the Degree of

Master of Science

in

Civil Engineering

Lehigh University

Bethlehem, Pennsylvania

June 1978

This thesis is accepted and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

May 3, 1978
Date

Professor in Charge

Chairman of Department

ACKNOWLEDGMENTS

The report herein, is related to the Lehigh University Project No. 402, which was conducted in Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. Dr. D. A. VanHorn is the Chairman of the Department. Dr. Lynn S. Beedle is the Director of the Fritz Engineering Laboratory. Dr. T. Huang is the project director.

The author sincerely appreciated and gratefully acknowledged his advisor, Dr. Ti Huang, for his supervision and patience in reviewing the author's thesis.

Sincere thanks are also due to Mrs. Ruth Grimes, who typed this manuscript.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

A generalized method for the prediction of prestress losses in pre-post-tensioned concrete members is presented in this thesis. Existing procedures for pretensioned and post-tensioned members have been modified and combined. Added flexibility allows the user to obtain the prestress losses in pre-post-tensioned as well as pre- and post-tensioned members. With minor adjustments, the presented procedures can also be used to analyze segmental members.

The stress-strain-time equations used in the existing procedures were found to provide a good model for the pretensioned case. The equations were modified to allow for multiple applications of additional concrete stresses at different times. By providing these additional capabilities, the procedure is made more powerful since it can now handle most real life situations. Another benefit of the new procedure is the ability to compute the steel stress in each level of steel in individual tendons while the existing procedure treats all prestressing steel as concentrated at c.g.s.

The procedure presented here is valuable due to its ability to handle more complicated members, and also its ability to provide more detailed results.

1. INTRODUCTION

1.1 Background

The study of prestress loss has been a continual research effort at Fritz Laboratory for many years. Project 339, Prestress Losses in Pretensioned Concrete Structural Members, was the initial project which was conducted from 1966 to 1972. In this project, the basic characteristics of concrete and steel materials were studied, and equations describing their respective stress-strain-time relaionships were developed. The material behavior equations were then connected by continuity and equilibrium equations to form a mathematical model for the prediction of prestress loss in pretensioned members.

Starting in 1972, a second study entitled "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams" (Fritz Laboratory Project 382) was undertaken. During this investigation, modifications were made to the previously developed concrete stress-strain-time relationship to reflect the behavior of pretensioned members more closely when additional load is applied after transfer. Observations from an experimental bridge were used to verify this modified procedure.

Project 402, the present project, was undertaken starting in 1972. The purpose of this project is to extend the previously developed procedure, so that it can be applied to post-tensioned as well as pre-post-tensioned members to predict their prestress losses.

1.2 Purpose

This thesis was undertaken to review all previous work on pre- and post-tensioned members, and to extend the prediction procedure to be applicable to pre-post-tensioned members. To accomplish this goal, a computer program FOUR02 was developed to provide a more generalized mathematical model. By providing additional generality the user is enable to treat problems involving the post-tensioning of members in several stages, the use of different types of steel strands in the same member, the application of additional loadings at several given times, and the variation of the level of the strands. These flexibilities also will permit the treatment of segmentally constructed members.

2. DEFINITIONS

2.1 Components of Prestress Loss

Several factors contribute to the amount of prestress loss. These factors will be briefly discussed.

Anchorage and Friction Losses

Only post-tensioned strands are affected by anchorage and friction losses. These losses are dependent on the slippage or deformation of the anchoring devices, the geometry of the strand profile, and the coefficient of friction between the strands and the concrete. These losses occur at the time the strand is tensioned and anchored, and vary in magnitude along the length of the member.

Elastic Shortening Loss

As the concrete is placed under a compressive load, the concrete shortens, permitting the steel also to shorten, thereby decreasing the steel stress. In the pretensioned member this occurs as the strands are released from the prestressing bed. Post-tensioned strands incur elastic loss only if and when additional strands are tensioned at a later time. The magnitude of the elastic loss is dependent primarily on the increment of concrete stress.

Shrinkage Loss

As concrete ages it losses moisture and shortens, causing a loss in prestress. Shrinkage starts at the time when curing of concrete is stopped and continues for the life of the member. This loss is dependent most importantly on the time after curing, the composition of concrete, and the volume-surface ratio of the member.

Creep Loss

Creep is defined as long term deformation of concrete under sustained compressive loading. As long as the load is maintained the concrete strain will continue to increase; therefore, creep is time dependent. Other important factors controlling creep loss include long-term concrete stress, composition of concrete, and the volume-surface ratio of the member, etc.

Relaxation Loss

Steel relaxation is similar to the creep of concrete, except that relaxation refers to a state of sustained strain. Under constant strain, the steel stress will decrease, or "relax," with time. Similar to creep, the rate of relaxation loss is greater under a larger initial stress (or strain).

The last two components of prestress loss, creep and relaxation, are not independent to each other. Both losses are dependent upon the magnitude of the stress in the respective material.

As prestress loss develops with time, both steel and concrete stresses are changed, and the rates of creep and relaxation are correspondingly affected. In a similar manner, shrinkage affects both the creep and the relaxation losses.

Another factor which affects prestress loss should also be mentioned here, that is the applied permanent loadings. Although loading of a member does not directly change the prestress, it does affect the concrete and the steel stresses. These changes in long-term stresses alter the rates of creep and relaxation, which in turn results in a change of the final prestress loss.

2.2 Description of Frabrication Procedures

Before explaining the equations which will be used to describe the behavior of prestressed concrete beams, it is appropriate to briefly review the basic procedures used in their fabrication and construction.

In the pretensioned fabrication, the strands are first stretched and anchored to a prestressing bed, then the concrete is placed. After the concrete has attained sufficient strength, the strands are released from the bed, placing the concrete in compression. The relaxation of the strands in the pretensioned member starts at the time of tensioning and anchoring to the bed. The elastic loss occurs during the release of the strands and the transfer of stress. Both shrinkage and creep can be considered to start at this same time.

A great advantage of post-tensioned fabrication is that it does not require a prestressing bed. First, the concrete member is cast. Prestressing tendons are placed in prepositioned conduits and are not bonded to concrete. The steel is tensioned while the concrete is compressed by jacking the steel against the end of the member, and a permanent prestressing bed is not needed. The friction between the steel and the conduit during tensioning, and the deformation of the anchorage devices immediately after, cause losses in steel stress in addition to the other common components. In a post-tensioned member, shrinkage of concrete starts at the termination of curing before prestressing takes place. However, the shrinkage occurring before post-tensioning does not cause loss of prestress. Both creep and relaxation start at the tensioning of the first strands.

By taking two or more pretensioned members and post-tensioning them together, we obtain a pre-post-tensioned member. As in the pretensioned case there is an elastic loss during transfer and the relaxation of the pretensioned strands start before the shrinkage and the creep of concrete. Upon post-tensioning, the pretensioned strands incur additional elastic loss due to the additional compression of concrete. However, this loss of prestress is counteracted by a stress gain caused by the additional dead load moment associated with the longer length.

Segmental construction may be viewed as an extension of the pre-post-tensioning procedure. Here short segments, which may or may not be pretensioned, are post-tensioned together one after another. The major difference between the pre-post-tensioned and the segmental members is that the segmental construction involves several post-tensioning stages when the number of steel strands and the dead weight moment change.

By understanding the type of problems that are to be handled, the formulation of the computational procedure is made easier. There are numerous differences among the four fabrication procedures, and to be applicable to all of them a very generalized procedure is needed.

3. DEVELOPMENT OF THE GENERALIZED PROCEDURE

From previous research conducted at Lehigh University, a stress-strain-time relationship was developed to describe the behavior of the steel strands[3].

$$f_s = f_{pu} \{A_1 + A_2 S_s + A_3 S_s^2 - [B_1 + B_2 \log (t_s + 1)] S_s$$
$$-[B_3 + B_4 \log (t_s + 1)] S_s^2\} \qquad (3\text{-}1)$$

where f_s = steel stress, in ksi (tension positive)

f_{pu} = specified ultimate tensile strength of steel, in ksi

S_s = steel strain, in 10^{-2} in/in. (elongation positive)

t_s = steel time, starting from tensioning, in days

The terms with A coefficients in Eq. 3-1 represent the instantaneous elastic response of the steel. Relaxation is accounted for by the B terms. Values for the steel coefficients have been empirically determined and are given in Table 1.

A similar equation was developed to represent the stress-strain-time relationship of concrete. This equation was subsequently modified in Fritz Laboratory Report 402.3[4] as follows:

$$S_c = -C_1 f_c + [D_1 + D_2 \log (t_c + 1)] + [E_1 + E_2 \log (t_c - t_{si} + 1)]$$
$$-f_c E_3 - E_4 (f_c - \Sigma f_{sdi}) \log (t_c - t_{si} + 1)$$
$$-E_4 \; \Sigma [f_{sdi} \log (t_c - t_{si} + 1)] \qquad (3\text{-}2)$$

where S_c = concrete strain, in 10^{-2} in/in. (shortening positive)

f_c = concrete stress, in ksi (tension positive)

t_c = concrete time, from end of curing, in days

t_{si} = concrete time at which the i^{th} increment of concrete stress, f_{sdi}, is applied, in days

f_{sdi} = i^{th} increment of concrete stress, in ksi (tension positive). This increment may be caused by either a tensioning of steel or additional loading, or both.

Here the C term represents the elastic response, the D terms the shrinkage, and the E terms the creep, values for the concrete coefficients are given in Table 2.

Now that the behaviors of the materials have been defined, the stress-strain-time relationships are linked together to describe the behavior of the whole member. Four linking equations are used. Relating the concrete and the steel times for each strand, we obtain

$$t_s = t_c - t_{si} \qquad (3\text{-}3)$$

For the pretensioned strands t_{si} will be negative, and t_s will be larger than t_c. Noting that the sum of the concrete strain (in compression) at the level of any strand and the tensile strain in that strand must remain constant at all times,

$$S_{si} + S_{ci} = k_{4i} \qquad (3\text{-}4)$$

where S_{si} = strain in the i^{th} strand, tension positive, in 10^{-2} in/in.

S_{ci} = concrete strain at the level of the i^{th} strand

k_{4i} = constant defined by the design and fabrication procedure of the member

Two more linking equations are used to insure equilibrium of the internal stresses.

$$-\int f_c dA_c - \Sigma f_{si} A_{si} = P \quad (3\text{-}5)$$

$$-\int f_c \, x \, dA_c - \Sigma f_{si} x_i A_{si} = -M \quad (3\text{-}6)$$

where A_c = area of net concrete section, in in.^{2}

A_{si} = area of steel in the i^{th} tensioning in in.^{2}

x = distance of differential area from the centroid of the net concrete section, in in.

P = applied axial load on section, in kips

M = applied bending moment on section, in kip-in.

The positive direction of x, P, and M are shown in Fig. 1.

Prior to further derivation, the equations will be simplified. Equation 3-1 can be rewritten as follows for each strand:

$$f_{si} = P_{1i} + P_{2i} S_s + P_{3i} S_s^2 \quad (3\text{-}7)$$

where $P_{1i} = A_1 f_{pu}$

$$P_{2i} = \{A_2 - B_1 - B_2 \log (t_c - t_{si} + 1)\} f_{pu}$$

$$P_{3i} = \{A_3 - B_3 - B_4 \log (t_c - t_{si} + 1)\} f_{pu}$$

Equation 3-2 can be simplified to

$$S_c = Q_1 - Q_2 f_c \qquad (3\text{-}8)$$

where $Q_1 = D_1 + D_2 \log (t_c + 1) + E_1 + E_2 \log (t_c - t_{s1} + 1)$

$$- E_4 \Sigma f_{sdi} [\log (t_c + 1 - t_{si}) - \log (t_c - t_{s1} + 1)]$$

$$Q_2 = C_1 + E_3 + E_4 \log (t_c - t_{s1} + 1)$$

Substituting Eq. 3-8 into Eq. 3-4 we obtain

$$S_{si} = k_{4i} - Q_1 + Q_{2fc} \qquad (3\text{-}9)$$

This relationship is substituted into Eq. 3-7, obtaining the following

$$f_{si} = P_{1i} + P_{2i} (k_{4i} - Q_1 + Q_2 f_c) + P_{3i} (k_{4i} - Q_1 + Q_2 f_c)^2$$

or $$f_{si} + R_{1i} + R_{2i} f_c + R_{3i} f_c^2 \qquad (3\text{-}10)$$

where $R_{1i} = P_{1i} + P_{2i} (k_{4i} - Q_1) + P_{3i} (k_{4i} - Q_1)^2$

$$R_{2i} = Q_2 [P_{2i} + 2P_{3i} (k_{4i} - Q_1)]$$

$$R_{3i} = P_{3i} Q_2^2$$

It is assumed that the concrete stress varies linearly across the depth of the cross-section, therefore;

$$f_c = g_1 + g_2 x \qquad (3\text{-}11)$$

This assumption implies that at any time when stresses are changed by post-tensioning or loading, the increments in concrete stresses will also be distributed linearly across the member. The change in concrete stress can be expressed as

$$f_{sdi} = f_{c\ after} - f_{c\ before}$$

$$f_{sdi} = (f_{1a} + g_{2a}\ x_i) - (g_{1b} + g_{2b}\ x_i)$$

or

$$f_{xdi} = H_{1i} + H_{2i}\ x_i \qquad (3\text{-}12)$$

where $H_{1i} = g_{1a} - g_{1b}$

$H_{2i} = g_{2a} - g_{2b}$

$g_{1a} = g_1$ after additional load is applied

$g_{2a} = g_2$ after additional load is applied

$g_{1b} = g_1$ before additional load is applied

$g_{2b} = g_2$ before additional load is applied

The significance of Eq. 3-12, and its use in the calculation procedure, is given in Section 4.4.

Rewriting Eq. 3-5 by introducing Eq. 3-11, we get

$$P = -g_1 A_c - \Sigma f_{si} A_{si}$$

$$P = -g_1 A_c - \Sigma [R_{1i} + R_{2i} (g_1 + g_2 x_i) + R_{3i} (g_1 + g_2 x_i)^2] A_{si}$$

or $$U_1 + U_2 g_1 + U_3 g_2 + U_4 g_1^2 + U_5 g_1 g_2 + U_6 g_2^2 = 0 \qquad (3\text{-}13)$$

where $U_1 = \Sigma R_{1i} A_{si} + P$

$U_2 = A_c + \Sigma R_{2i} A_{si}$

$U_3 = \Sigma R_{2i} x_i A_{si}$

$U_4 = \Sigma R_{3i} A_{si}$

$U_5 = 2 \Sigma R_{3i} x_i A_{si}$

$U_6 = \Sigma R_{3i} x_i^2 A_{si}$

Likewise Eq. 3-6 can be rewritten in terms of g_1 and g_2 to obtain

$$-M = -I g_2 - \Sigma f_{si} A_{si} x_i$$

$$M = I g_2 + \Sigma [R_{1i} + R_{2i} (g_1 + g_2 x_i) + R_{3i} (g_i + g_2 x_i)^2] x_i A_{si}$$

or $$V_1 + V_2 g_1 + V_3 g_2 + V_4 g_1^2 + V_5 g_1 g_2 + V_6 g_2^2 = 0 \qquad (3\text{-}14)$$

where $V_1 = \Sigma R_{1i} A_{si} x_i - M$

$V_2 = \Sigma R_{2i} A_{si} x_i = U_3$

$V_3 = I + \Sigma R_{2i} A_{si} x_i^2$

$V_4 = \Sigma R_{3i} A_{si} x_i = \frac{1}{2} U_5$

$V_5 = 2 \Sigma R_{3i} x_i^2 A_{si} = 2 U_6$

$V_6 = \Sigma R_{3i} x_i^3 A_{si}$

In the above derivations all summations cover all strands which have been attached to concrete at the given time t_c, not involving the strands, if any, which are being post-tensioned at this time. At any given time, the U and V coefficients in Eqs. 3-13 and 3-14 can be computed, and the only unknowns remaining are g_1 and g_2. Having two equations and two unknowns, Eqs. 3-13 and 3-14 can now be solved simultaneously. In program FOURO2, Newton's method for nonlinear equations was used for this purpose. After the solution of g_1 and g_2, the concrete and the steel stresses can be easily determined from Eqs. 3-10 and 3-11.

4. DISCUSSION

4.1 Location of Critical Section

Structural design of any member is usually controlled by the conditions at a certain cross-section. For the simply supported beam, the midspan is taken as the critical cross-section. Here the moments due to dead and live loads are at the maximum. Also, the anchorage and friction losses tend to be the greatest at this section, hence the prestress tends to be the smallest, if the strands are jacked from both ends. Because of these conditions, program FOUR02 only calculates the stresses at the midpoint of the beam. However, it can easily be modified to accommodate stress evaluations at other locations.

The dead weight moment, which is largest at the midpoint of the beam, is assumed to be acting on the section upon release of the pretensioned strands. The assumption which was made here is that there is sufficient prestressing steel present to place the concrete at the midpoint bottom of the beam in compression, causing the beam to camber upward. This condition is almost always present, in reality, hence this assumption does not limit the usefulness of the procedure in any practical sense.

4.2 Anchorage and Friction Losses

For a post-tensioned strand, there are two prestress loss components which occur during the tensioning time, namely: anchorage loss and friction loss. Both of these losses will reduce the actual steel stress at an interior section below the jacking stress measured at the end or ends of the member. The exact derivations of the anchorage and friction losses used in program FOUR02 has previously been presented in Fritz Laboratory Report 402.34, and will not be repeated here. A brief summary will suffice.

Friction loss is a result of having the steel move relative to the concrete during tensioning. This movement is resisted by the friction between the steel and the inside of the conduit causing steel stress to decrease away from the jacking end. Afterwards, as the anchoring device takes effect, its deformation or displacement allows the steel tendon to slide inward. This motion is again resisted by friction, and steel stress decreases toward the anchored end. If the anchorage displacement is large, or the friction coefficients are small, the inward movement of steel may penetrate the entire length of the member, and will control the steel stress everywhere. Otherwise, the friction loss may control the stress at the critical location. Figure 4 shows a typical stress distribution in a beam which was tensioned from two ends and where friction losses control.

A fact that should be mentioned is that the anchorage and friction losses are location dependent, and not time dependent. In program FOUR02 these initial losses are computed at the midpoint of the beam. The steel stress immediately after tensioning, after anchorage and friction losses, is defined as the initial steel stress. This stress is used as the basis for all future calculations.

4.3 Shrinkage and Creep

The empirically derived concrete material equation (Eq. 3-2) implies a non-zero shrinkage strain at $t_c = 0$, in the magnitude of $-D_1$. This term was included in the regression analysis expression in the interest of improved long term prediction. To insure proper evaluation of stresses at transfer for pretensioned members, this term is ignored for $t_c < 1$ day. In effect, a discontinuity is created at $t_c = 1$ day, which is judged small enough to be tolerated. Likewise, the E_1 and E_3 terms in the creep strain expression are ignored for the first day following the first post-tensioning ($t_c \leq t_{s1} + 1$) to insure accuracy in elastic stress calculations. The delay period of one day is arbitrarily chosen. The purpose for the one-day delay is to assure that all strands post-tensioned consecutively within a short time period will experience similar losses.

4.4 Refinement of Creep Strain

Equation 3-2 is a refined version of the concrete stress-strain-time relationship originally developed in Project 339. The refinement was needed to gradually phase in the creep effect caused by loads applied after transfer. The original equation, after adjustments to conform with the sign conventions used in this thesis, was

$$S_c = -C_1 f_c + [D_1 + D_2 \log (t_c + 1)] + [E_1 + E_2 \log (t_c + 1)] - f_c [E_3 + E_4 \log (t_c - 1)] \qquad (4\text{-}1)$$

This equation was previously modified by Hsieh, for the conditions after t_{c5} and an increment of stress f_{sd} is applied, as follows[1]

$$S_c = -C_1 f_c + [D_1 + D_2 \log (t_c + 1)] + [E_1 + E_2 \log (t_c + 1)] - f_c [E_3 + E_4 \log (t_c + 1)] + E_4 f_{sd} [\log (t_c + 1) - \log (t_c + 1 - t_{c5})] \qquad (4\text{-}2)$$

Comparing Eqs. 4-1 and 4-2, the modification involves changing the time parameter for the creep strain due to the stress increment. Hsieh reasoned that as an additional load is applied to a member, the added concrete stress causes additional creep only after its application. In Eq. 4-1, the increment in concrete stress, f_{sd}, is included in f_c, consequently it would produce creep as if it existed from the initial prestress time $t_c = 0$. This is clearly an overestimate of creep, and it actually causes a significant discontinuity in the strain at the time of load application. By adding the last term in Eq. 4-2, Hsieh removed the excess creep, and treated the additional stress as producing additional creep only for the time after its application. Figure 2 illustrates the effects of the modification.

Hsieh was able to determine the f_{sd} value by an analytical procedure as follows: For time $t_c = t_{c5}$, concrete strains and stresses before loading are related by Eq. 4-1, and those after loading by Eq. 4-2. The two relationships are subtracted to form a relationship between the changes in concrete strain and the changes in concrete stress. Using the linking equation for the strains, the change in the steel stress is also related to the changes in the concrete at strain at c.g.s. Placing these two relationships into the equilbrium equations, a single polynomial equation in terms of f_{sd} can be obtained. This equation is then solved for f_{sd}. The detailed development is given in Ref. 1.

The above technique was expanded by permitting multiple loadings. Additional creep terms are just superimposed on top of the existing creep terms, and regrouped, resulting in Eq. 3-2. Determination of f_{sdi} presents several minor problems. First, as multiple layers of prestressing steel are to be permitted, f_{sdi} must be defined for each strand level. To provide a continuous definition at every point, f_{sdi} was defined in Chapter 3

$$f_{sdi} = H_{1i} + H_{2i} x_i \qquad (3\text{-}12)$$

A second complication, as Hsieh noted, was that while f_{sd} was the difference between the stresses before and after the loading, evaluation of the stresses after loading is dependent on f_{sd}. Hsieh's direct solution method is impractical for the generalized case under consideration. In Program FOUR02, an iterative method is employed. The concrete stresses immediately before loading are directly calculated. The concrete stresses immediately after loading are initially calculated assuming all f_{sdi} to be zero. The difference between the two sets of concrete stresses yields an approximate estimate of f_{sdi}. These estimates of f_{sdi} enable the recalculation of the concrete stress after loading, which in turn yields improved estimates of f_{sdi}. The iteration procedure is repeated until f_{sdi} values are essentially stabilized. This is usually achieved in four or five cycles.

4.5 Post-Tensioning

When a tendon is post-tensioned the stress in the tendon is being controlled and known. The stresses (strains) in the tendons that have previously been attached to the members are not being measured, but must be calculated on the basis of elastic response of the "previous" transformed section which includes the concrete section and all previously attached strands. In program FOUR02, the post-tensioning is treated as an application of an equivalent eccentrical load on the "previous" section. The equivalent axial load is the initial tensile force in the tendons being stretched. The equivalent applied moment is the product of the initial force and the distance between the steel being post-tensioned and the centroid of the transformed section. Figure 5 helps to clarify this equivalent force system for post-tensioning. The transformed section properties are used instead of the net properties because the effects of previously anchored steel must be included. The equivalent external loads are added to the existing external loads. After the solution, the equivalent load system is removed, and the actual loading and steel areas are carried forward for calculations at later ages.

4.6 Composite Beam Loads

The effect of composite action of cast-in-place bridge deck is handled in the same manner as in the existing pretensioned and post-tensioned programs. Loads imposed on the composite section are converted into systems of external loads which when applied to the precast cross-section will result in identical stresses.

By following this procedure, program FOUR02 deals with only the precast cross-section, and not the composite section. For this reason, no strand may be tensioned after the placing of the deck, because any such post-tensioning will be resisted by the composite section. This limitation does not reduce the practical usefulness of the program, however; since the usual erection procedure would not involve post-tensioning after the deck is placed.

When only an additional moment is added to the beam, the procedure treats the loading as a ficticious tensioning of strands with an area equal to zero. By setting the steel area equal to zero, all contributions in Eqs. 3-13 and 3-14 from the steel area during this loading become zero. The loading will however directly effect the concrete stress equation since the f_{sdi} value corresponding to it will not be zero. An adjustment is made to the external loads to reflect the additional external moment.

4.7 Comments on Segmental Modeling

Although not presently designed to handle segmental construction, the program FOUR02 can be modified for additional application. The basic beam to be analyzed will be only one segment in length. Since the post-tensioning strands will be tensioned through several segments, the specified jacking stress would have to be reduced to account for the increased initial losses before entering the present program. As successive segments are added, additional dead load moments are also added to the beam to reflect the actual loading. By using this approach a segmental beam could be accurately modeled, although the calculations would be considereably more involved than the present case.

4.8 Description of Example Bridge

Further discussion of the generalized procedure is based on its application to an example beam member. The member is similar to an example used before in Project 339.9. It consists of a 31.39 m (103 ft.) beam of the AASHO Type IV I-beam cross-section prestressed with a total of 66-7/16 in. strands. The entire beam is topped with a 190 mm x 2.13 m (7.5 x 84 in.) deck 160 days after post-tensioning. (A detailed description of this beam is given in Appendix A.) This member is analyzed three times, taken as pretensioned, post-tensioned and pre-post-tensioned, respectively. For the pre-post-tensioned case this beam is assumed to be cast in three segments, two of 7.62 m (25 ft.) length and a central piece 16.15 m (53 ft.) long. The center piece is pretensioned by fifteen strands, which is adequate to counteract the dead load moment of the segment itself. After thirty days, the 7.62 m (25 ft.) segments are attached to the ends of the pretensioned segment by 51 post-tensioned strands, all of which are stretched at the same time.

In the pretensioned case all the tendons were tensioned to 0.75 f_{pu} three days prior to the placement of the concrete. The initial tensioning stress was chosen to be 0.75 f_{pu} to account for the elastic loss at release. Likewise, the pretensioned steel in the pre-post-tensioned member was tenstioned to .71 f_{pu}. These tensioning stresses were chosen so that immediately after release, these strands will have stress of 0.70 f_{pu}, same as the tensioning stress of post-tensioned strands. The pretensioned member is fabricated in its full length of 31.39 m (103 ft.).

For the post-tensioned case all sixty-six strands were tensioned simultaneously thirty days after placement of concrete. The tensioning stress was 0.70 f_{pu}. Once again the full length of 31.39 m was used. All of the strands were straight with minimal values for friction, wobble, and anchorage losses.

4.9 Comparison of Pre-, Post-, and Pre-Post-Tensioned Cases

The example beam that was described in Section 4.8 was analyzed three times using program FOURO2. In the three computer runs, the beam was treated as pretensioned, post-tensioned, and pre-post-tensioned members, respectively. The results of these analyses are given in Table 3 and Fig. 3.

As was expected, the strands in the pretensioned beam and the pretensioned strands in the pre-post-tensioned beam had the lowest stress at the end of the assumed useful service life (36500 days). Initially, the strands in the pretensioned beam were tensioned to a higher stress to accommodate the elastic loss at release. After elastic loss the stresses are both at 1228 MPa (178.2 ksi). In the pre-post-tensioned member, post-tensioning at thirty days caused an additional elastic loss in the pretensioned strands. Subsequently, these strands were at a lower stress than those in the pretensioned member. These comparisons are clearly shown in Fig. 3, as was expected.

Unexpectedly, it was noticed that the strands in the post-tensioned member had a higher calculated final stress than the post-tensioned tendons in the pre-post-tensioned member, 1006 MPa (145.9 ksi) vs. 997 MPa (144.5 ksi). At the time of post-tensioning, the pretensioned strands in the pre-post-tensioned member have already lost a fraction of their initial tensioning stress. Therefore, after post-tensioning, the concrete stresses in the member can be expected to be lower than in the post-tensioned member. This in turn should induce lower creep and hence less prestress loss. Therefore, the post-tensioned member is expected to have a lower final prestress. The computer results showed difficulty.

The unexpected comparison of the two members is seen to start at the time of post-tensioning. To further pinpoint the origin of this unexpected behavior, additional computer runs were made with reduced time intervals immediately following post-tensioning. The results are shown in Table 3.

At one-half day after post-tensioning (t_c = 30.5) the steel stresses in the two members are approximately equal, 1238 MPa (179.6 ksi) vs. 1241 MPa (179.9 ksi). The pre-post strand however, seemed to have a lower rate of stress loss than the post-tensioned strand. This is in line with the reasoning above. This same trend continues for another half day. At t_c = 31 days, the post-tensioning steel stresses were 1229 MPa and 1232 MPa in the two members. At 1-1/2 day after tensioning, t_c = 31.5, a discontinuity occurs in the post-tensioned member. Instead of a loss, the steel stress showed a gain of approximately 15 MPa (2 ksi) in the half-day period. The steel stress in the pre-post-tensioned case has continued to decrease as was expected. At this time, the stress in the post-tensioned member is 19 MPa (2.68 ksi) higher. This same difference remained practically unchanged to the end of 100 years. It is apparent therefore, that the cause of the discrepancy took place at $t_c = 31.5$ days. Upon closer examination, the jump in the steel stress is seen to be caused by the discontinuity in the concrete stress-strain-time relationship, as was previously mentioned in Section 4.3. The E_1 and E_3 terms for creep are not included until one day after prestressing of concrete. In the post-tensioning case, the inclusion of these terms for the first time at $t = 31.5$ days created the stress increase. For the pre-post-tensioned case, these terms are included from one day after pretension transfer, at $t_c = 1.0$ day, hence no discontinuity was introduced after the post-tensioning.

As explained previously (Section 4.4), the one day delay in applying the terms E_1 and E_3 was introduced so that several strands stretched within a short period of time will be treated alike. Without a delay time, the strands stretched first will incur a disproportional loss and appear to behave differently from all other strands, which is clearly unacceptable. The time interval by which these terms are delayed was chosen more or less arbitrarily.

However, a change in the delay time will only change the time of the discontinuity, but not the final result. Bearing in mind that the concrete stress-strain-time relationship was derived empirically with attention focused on long term stability, some loss of short term accuracy is tolerated. In the present case, the calculated stresses in the post-tensioned member at t_c = 31.5 days may be slightly in error, however; the long-term predictions are expected to be reasonably accurate. The difference between the two members amounts to approximately 1% of the guaranteed strength. This discrepancy is very small in comparison to other uncertainties inherent in the procedure regarding the material quality and location. The magnitude of this discrepancy will be affected by the amount of pretension in the pre-post-tensioned member. This example problem, with a small amount of pretensioning would show a high discontinuity effect. Therefore, it can be reasoned that the error induced by this discontinuity is generally very small, and further refinement is not justified.

5. CONCLUSIONS AND RECOMMENDATIONS

1. The computer program FOURO2 can be used to estimate prestress losses in pretensioned, post-tensioned, and pre-post-tensioned members.

2. Program FOURO2 can also be used to estimate the prestress losses in segmental members by modifying the input data to represent an equivalent section.

3. A discontinuity in the concrete stress-strain-time relationship causes a discontinuity in the computed stresses one day after the first prestressing. This discontinuity does not affect long-term prediction, and is usually of a small magnitude. Therefore, it is judged acceptable.

4. Futher investigations should be undertaken to compare the pre-post-tensioned and the segmental predictions with actual field data. This would test the theoretical results against the actual conditions.

6. TABLES

TABLE 1 COEFFICIENTS FOR STEEL SURFACES

Instantaneous Stress-Strain Relationship					
All Sizes	All Manufacturers	$A_1 = -0.4229$ $A_2 = 1.21952$ $A_3 = -0.17827$			
Relaxation Coefficients - Stress Relieved Strands					
Size	Manufacturers	B_1	B_2	B_3	B_4
7/16 in.	B	-0.05243	0.00113	0.11502	0.05228
	C	-0.04697	-0.01173	0.10015	0.05943
	U	-0.06036	0.00891	0.12068	0.02660
	ALL	-0.05321	0.00291	0.11294	0.03763
1/2 in.	B	-0.06380	0.00359	0.12037	0.05673
	C	-0.07880	-0.00762	0.14598	0.05920
	U	-0.06922	0.00844	0.13645	0.04[illegible]4
	ALL	-0.07346	0.00620	0.13847	0.04608
ALL	ALL	-0.05867	0.00023	0.11860	0.04858
Low-Relaxation Strands					
7/16 in.	L	-0.00412	0.00142	0.02203	0.01605
1/2 in.	L	-0.02672	0.01399	0.04435	0.00923
	ALL	-0.01403	0.00609	0.03245	0.01395

TABLE 2 COEFFICIENTS FOR CONCRETE SURFACES

Coefficients		Upper Bound	Lower Bound	Combined
Elastic Strain C_1*		0.02500	0.02105	0.02299
Shrinkage	D_1	-0.00668	-0.00066	-0.00289
	D_2	0.02454	0.01500	0.02031
Creep	E_1	-0.01280	-0.00664	-0.01592
	E_2	0.00675	-0.00331	0.00649
	E_3	-0.0060	-0.00371	0.00256
	E_4	0.01609	0.01409	0.01153

*Note: $C_1 = 100/E_c$ where E_c is modulus of elasticity for concrete, in ksi

TABLE 3 RESULTS OF EXAMPLE CALCULATIONS

Concrete Time (Days)	Total Prestress Force in Steel							
					Pre-Post-Tensioned			
	Pretensioned Member		Post-Tensioned Member		Pretensioned Strands		Post-Tensioned Strands	
	MPa	(ksi)	MPa	(ksi)	MPa	(ksi)	MPa	(ksi)
-3	1365	(197.96)			1297	(188.15)		
0	1303	(188.99)			1245	(180.58)		
0	1228	(178.16)			1229	(178.22)		
2	1215	(176.27)			1223	(177.39)		
3	1206	(174.85)			1217	(176.56)		
5	1192	(172.83)			1209	(175.31)		
7	1182	(171.38)			1202	(174.39)		
10	1170	(169.77)			1195	(173.34)		
30	1135	(164.56)			1171	(169.79)		
30			1278	(185.37)	1121	(162.61)	1278	(185.37)
30.5			1238	(179.61)	1118	(162.15)	1241	(179.92)
31			1229	(178.25)	1114	(161.53)	1232	(178.67)
31.5			1244	(180.38)	1110	(161.03)	1225	(177.70)
32			1238	(179.49)	1107	(160.61)	1220	(176.90)
33			1232	(173.75)	1103	(159.94)	1211	(175.65)
50	1118	(162.11)	1175	(170.36)	1072	(155.41)	1161	(168.35)
100	1095	(158.83)	1136	(164.69)	1042	(151.10)	2283	(162.82)
190	1075	(155.87)	1109	(160.91)	1019	(147.34)	1097	(159.10)
190	1089	(157.89)	1123	(162.90)	1034	(149.93)	1111	(161.10)
300	1094	(158.60)	1126	(163.25)	1039	(150.66)	1114	(161.50)
500	1082	(156.90)	1112	(161.30)	1027	(148.96)	1100	(159.60)
1000	1065	(154.41)	1094	(158.60)	1010	(146.51)	1082	(156.96)
3000	1038	(150.50)	1065	(154.47)	934	(142.71)	1054	(152.92)
5000	1026	(148.74)	1052	(152.65)	972	(141.01)	1042	(151.13)
10000	1009	(146.40)	1036	(150.20)	957	(138.77)	1026	(148.74)
20000	994	(144.15)	1019	(147.35)	942	(136.00	1010	(146.45)
20500	931	(142.25)	1006	(145.32)	929	(134.73)	997	(144.53)

7. FIGURES

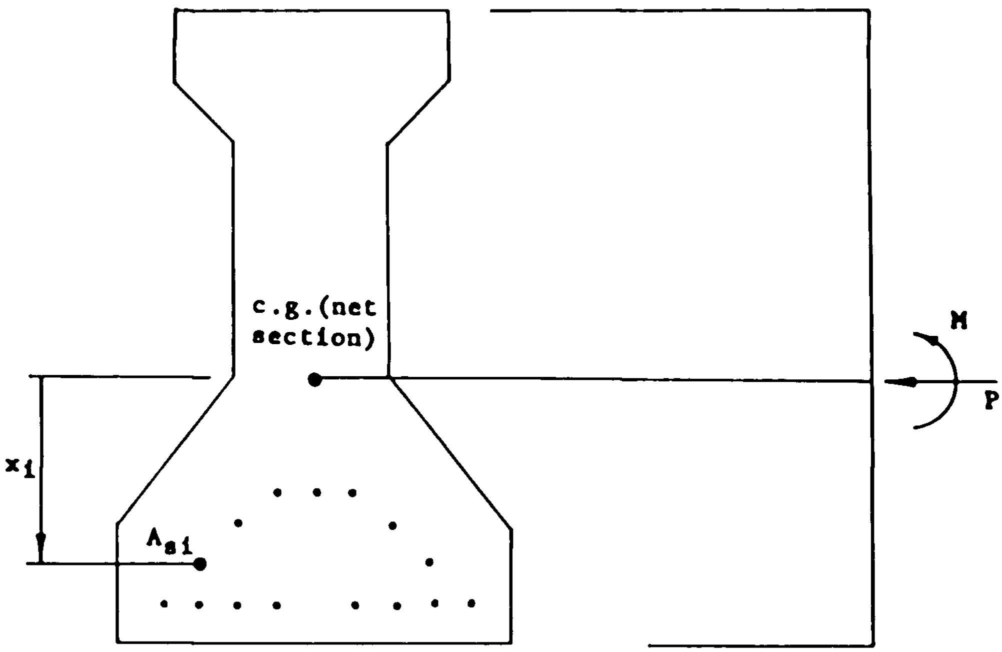


Fig. 1 Sign Convention for Applied Loads

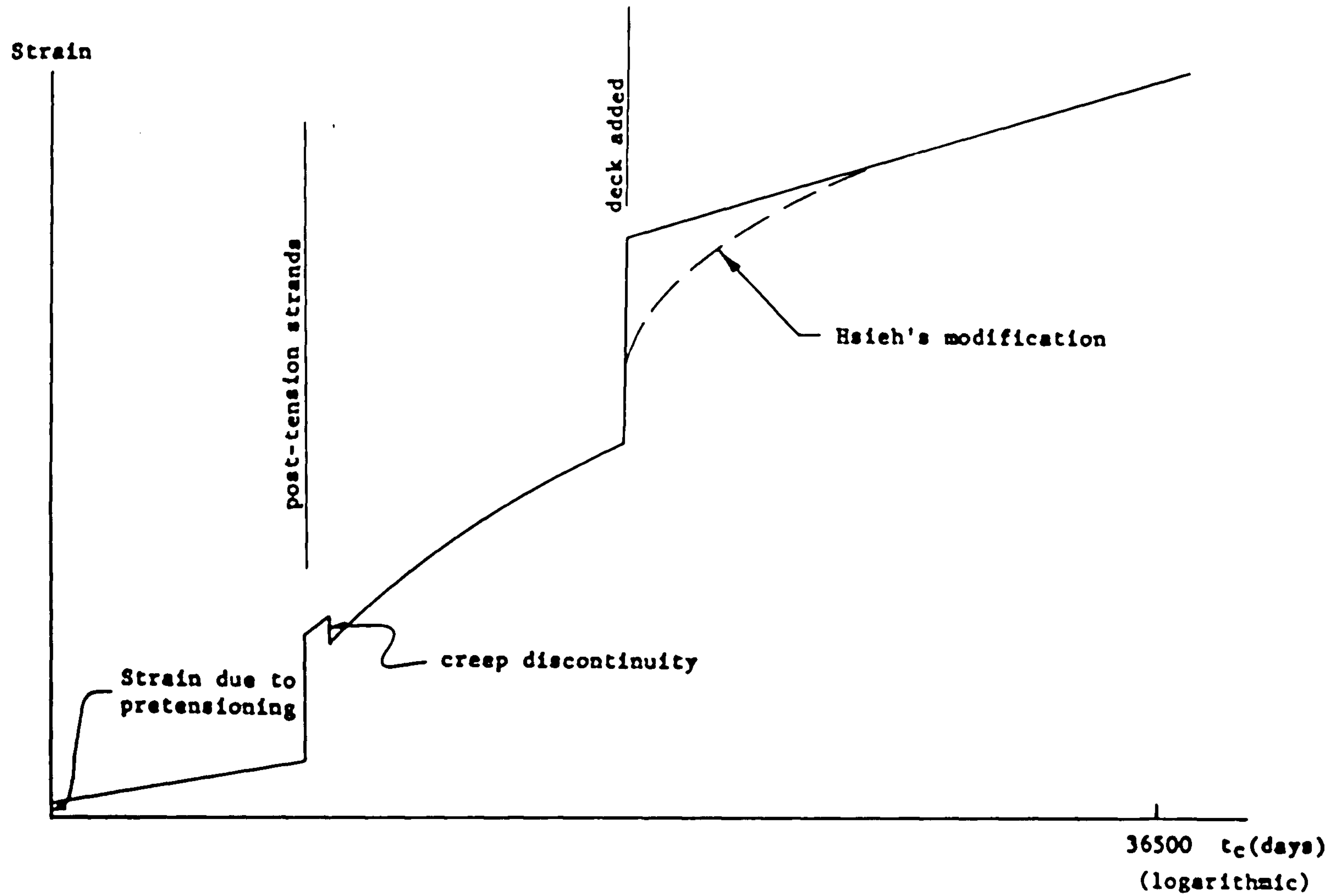


Fig. 2 Typical Concrete Strain in a Pre-post-tensioned Member

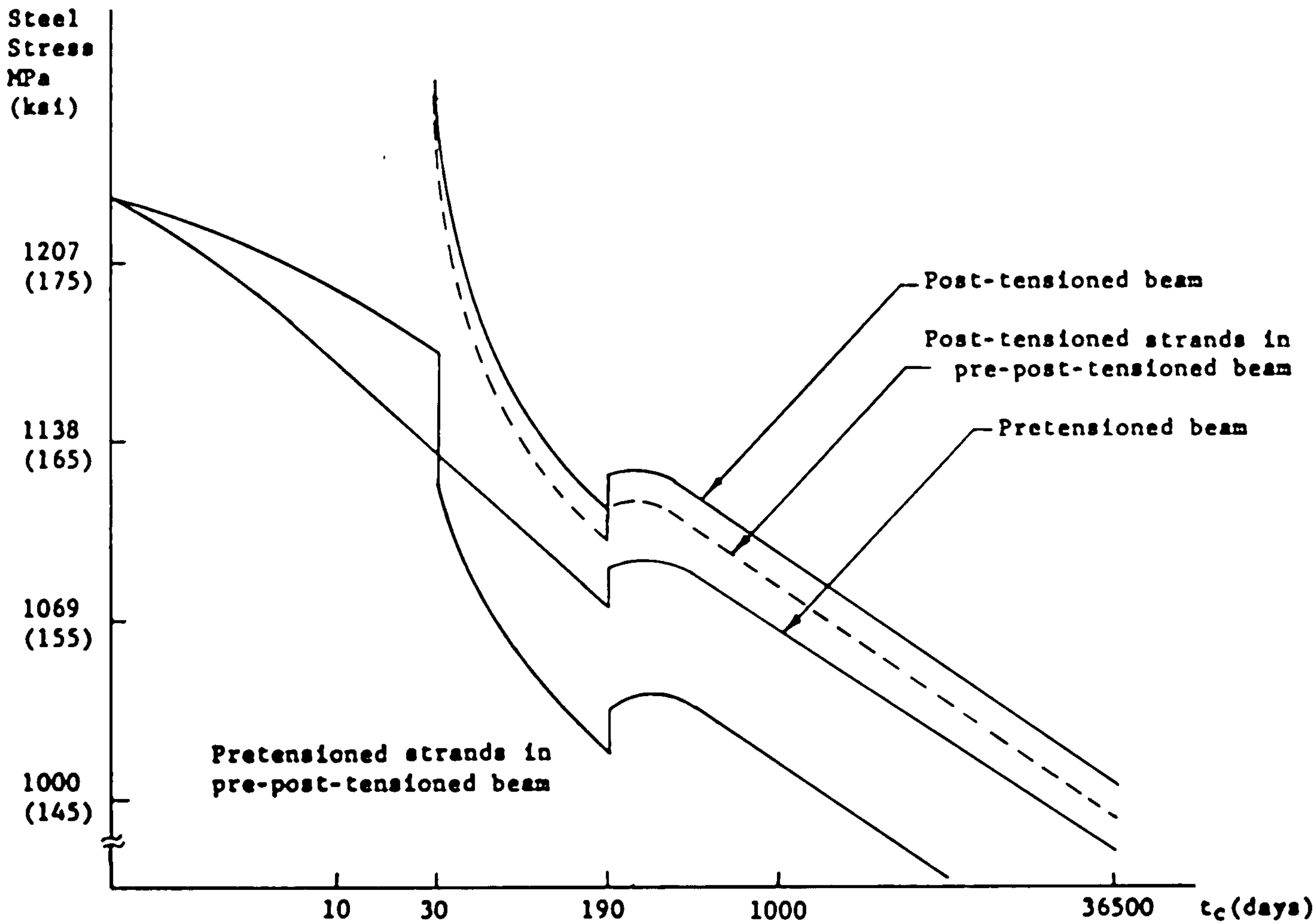


Fig. 3 Comparision of Three Example Cases

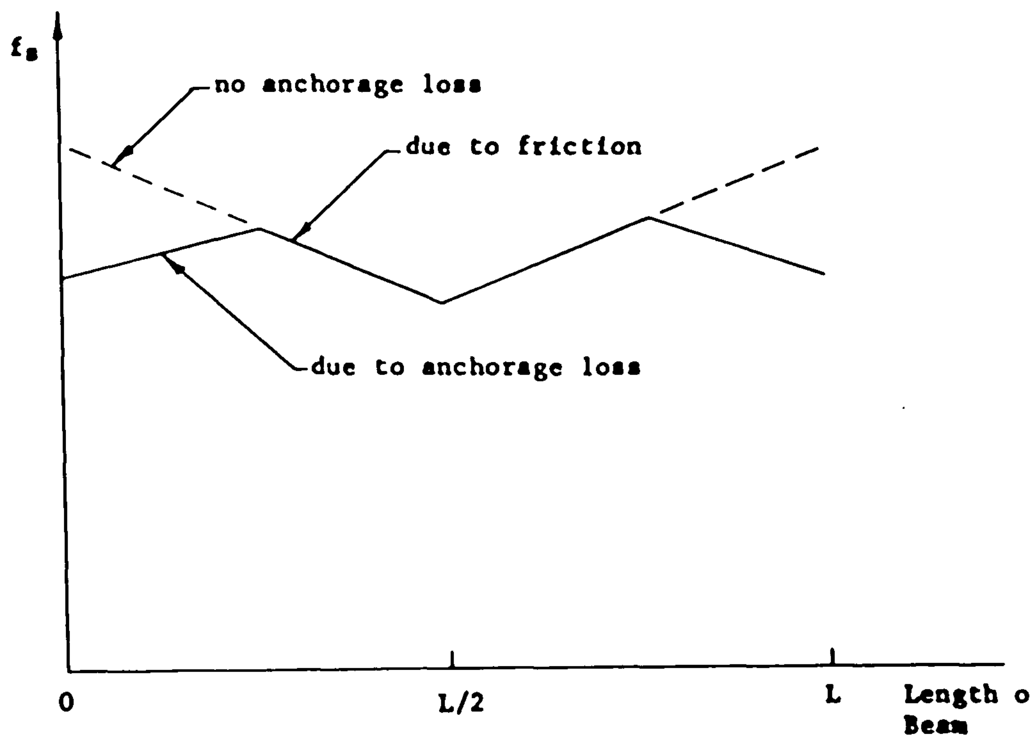


Fig. 4 Initial Losses in a Beam Jacked From Both Ends

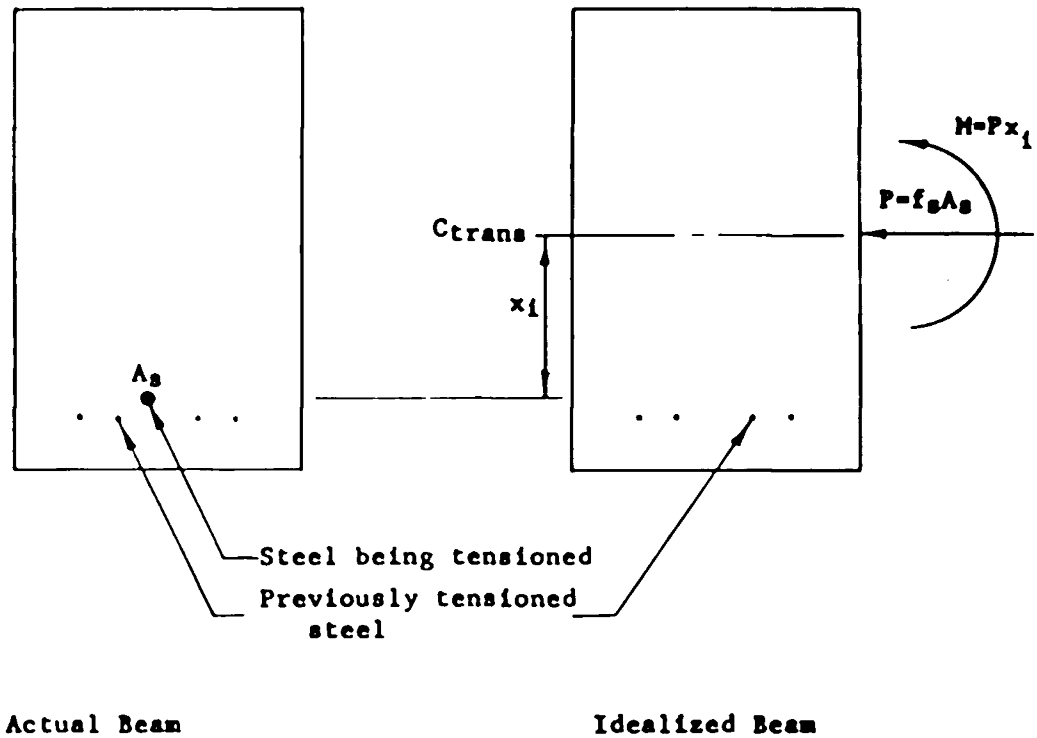


Fig. 5 Equivalent Loading for Post-tensioning

8. REFERENCES

1. Hsieh, Chung-Sing
IMPROVEMENT OF PREDICTION PROCEDURE FOR PRESTRESS LOSSES IN PRETENSIONED CONCRETE BEAMS, Masters' Thesis, Lehigh University, 1975.

2. Huang, Ti
ANCHORAGE TAKE-UP LOSS IN POST-TENSIONED MEMBERS, Journal of the Prestressed Concrete Institute, Vol. 14 (1969), No. 4, p. 30.

3. Huang, Ti
PRESTRESS LOSSES IN PRETENSIONED CONCRETE STRUCTURAL MEMBERS, Fritz Engineering Laboratory Report No. 339.9, Lehigh University, August 1973.

4. Huang, Ti and Hoffman, Burt
PRESTRESS LOSSES IN POST-TENSIONED CONCRETE STRUCTURAL MEMBERS, Fritz Engineering Laboratory Report No. 402.3, Lehigh University, to be issued.

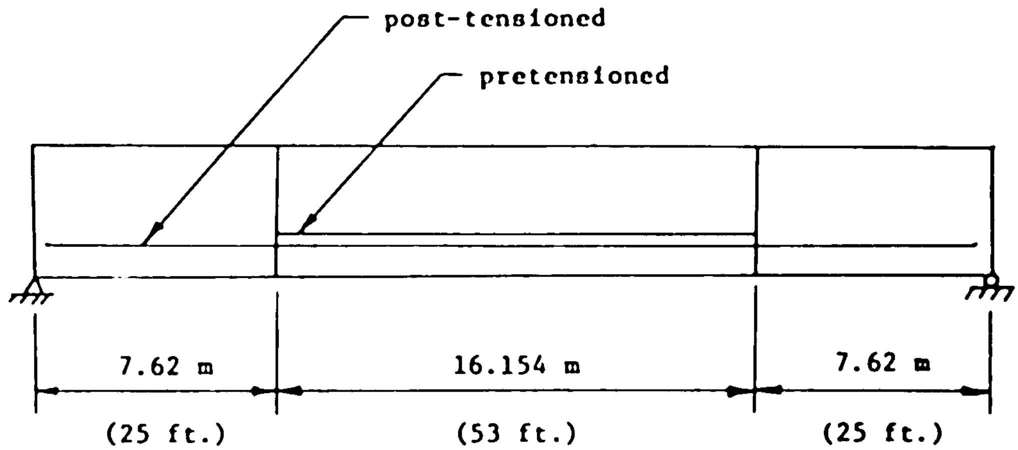
5. Lin, T. Y.
DESIGN OF PRESTRESSED CONCRETE STRUCTURES, Second edition, John Wiley and Sons, Inc., New York, 1963.

6. Tansu, John
PREDICTION OF PRESTRESS LOSSES FOR PRESTRESSED CONCRETE MEMBERS, Masters' Thesis, Lehigh University, 1975.

9. APPENDICES

APPENDIX A

Example Pre-Post-Tensioned Bridge Member



Type of strand = 7/16 in. diameter, stress-relieve strands, manufacturer C

Steel surface: 7/16 in. diameter stress relieved strands, all manufacturers

Concrete surface solution: Lower bound losses

Number of pretensioned strands = 15

Number of post-tensioned strands = 51

Net area of concrete = 0.5471 m^2 (848.0 in^2)

Moment of inertia of concrete net section = 0.1457 m^4 (350,000 in^4)

Depth of beam = 1.524 m (60 in.)

Level of steel = 1.30 m (51.2 in.) from top

Tensioning stress level; pretensioned - 0.71 f_{pu}

post-tensioned - 0.70 f_{pu}

Structural thickness of deck slap = 0.1778 m (7.0 in.)

Total thickness of deck = 0.1905 m (7.5 in.)

Effective width of deck = 2.134 m (84.0 in.)

Time of pretensioning = -3.0 days

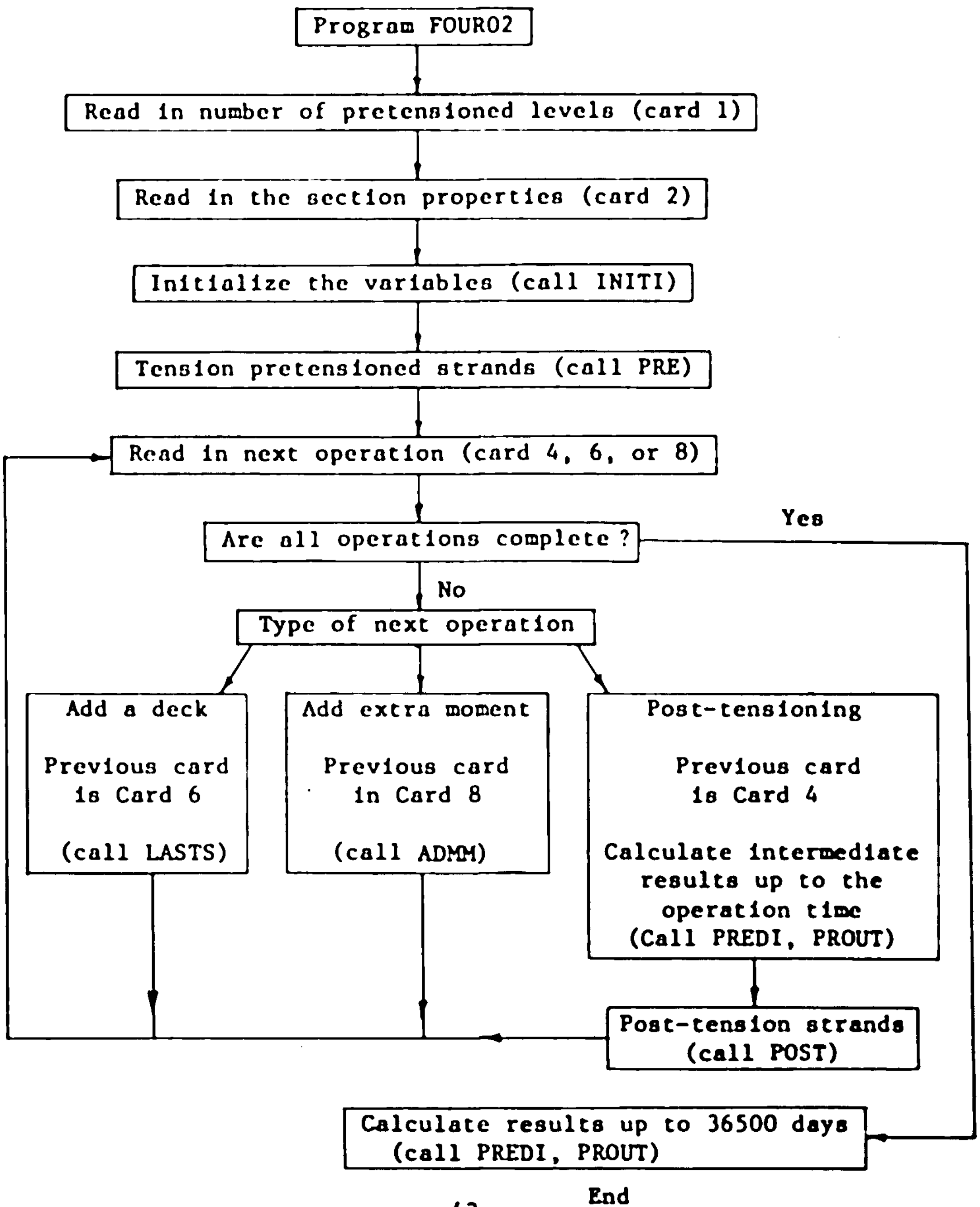
Time of post-tensioning = 30 days

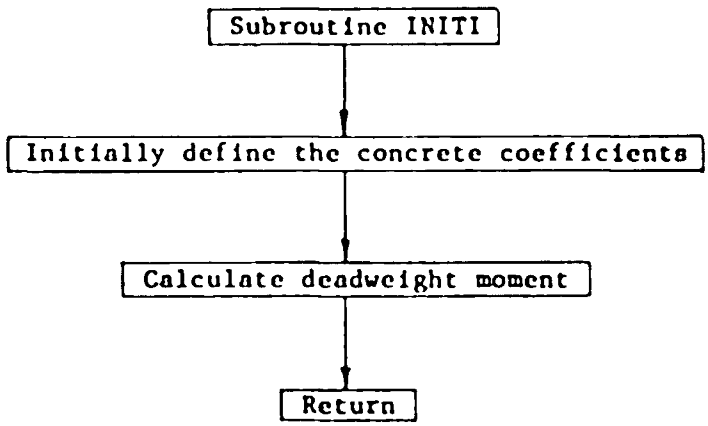
Time deck slab is cast = 190 days

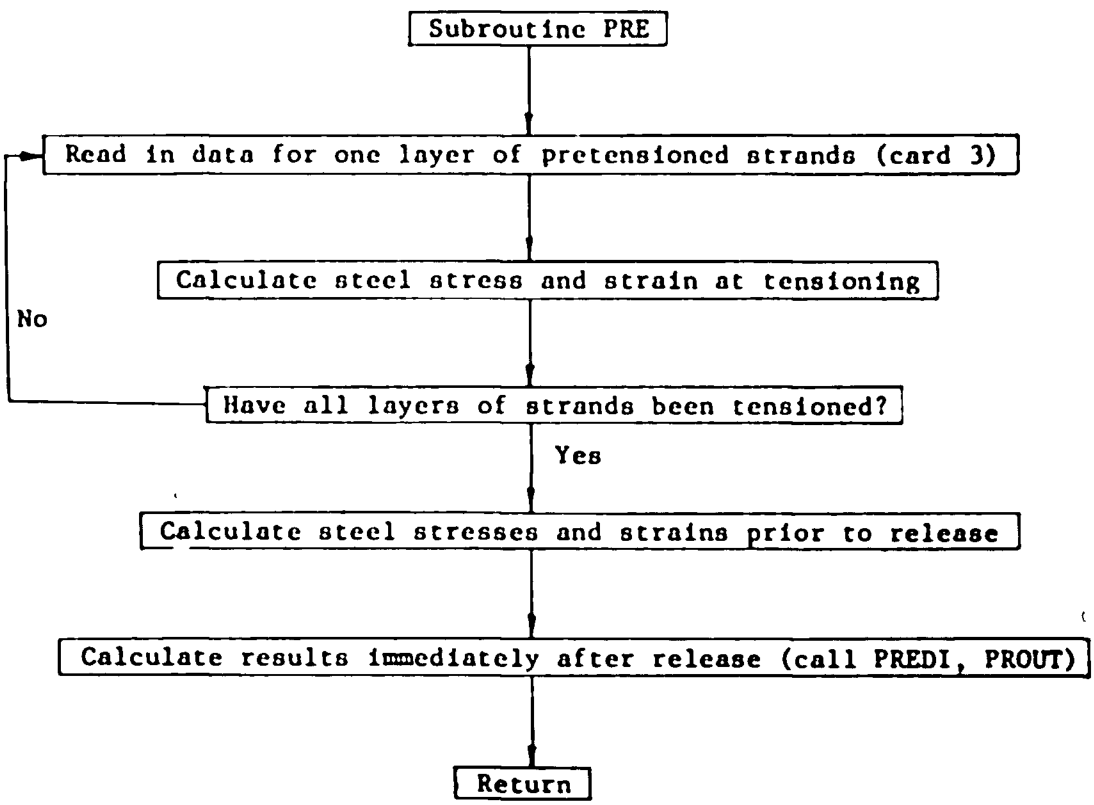
Anchorage and friction losses $\approx$ 0

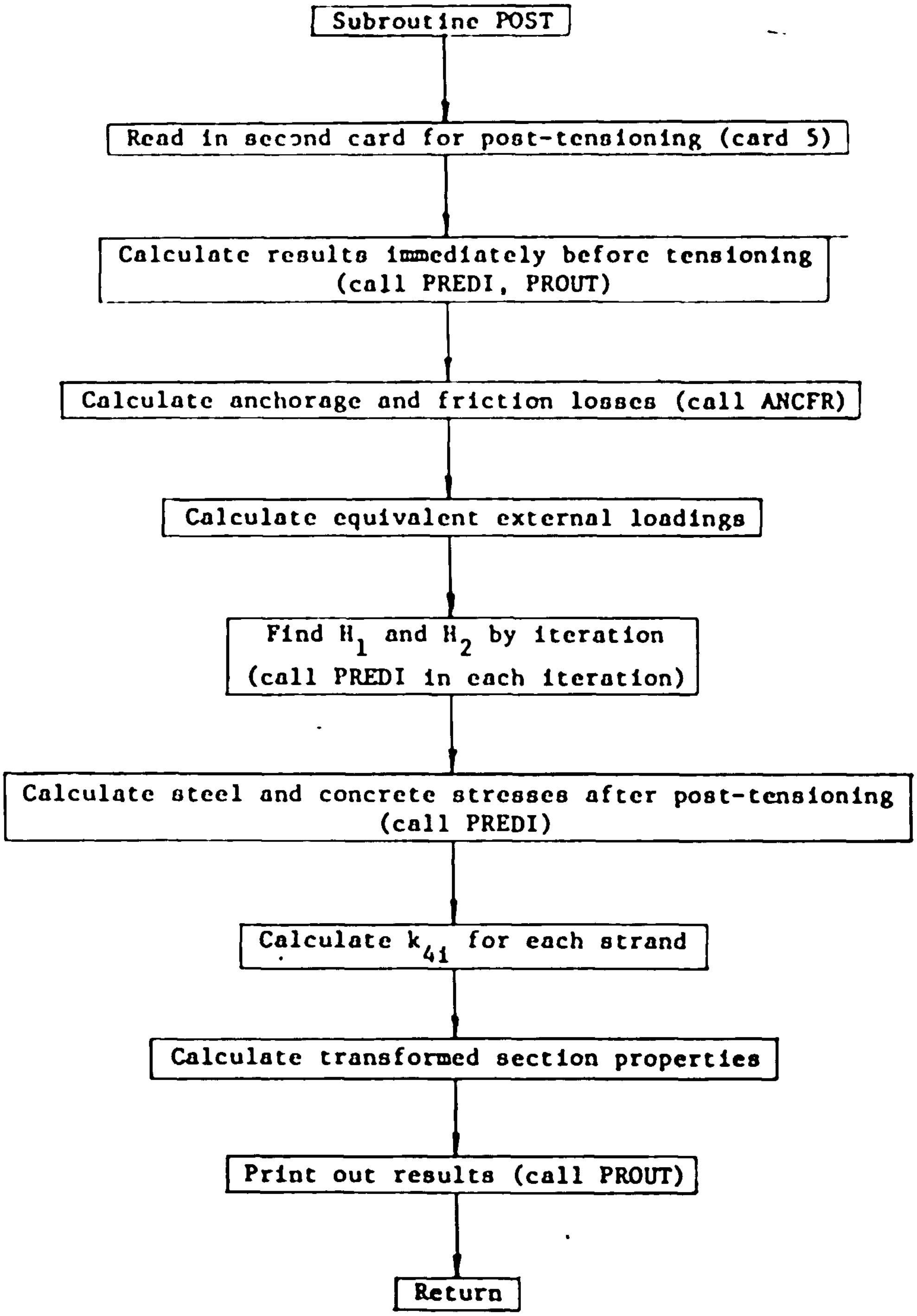
APPENDIX B

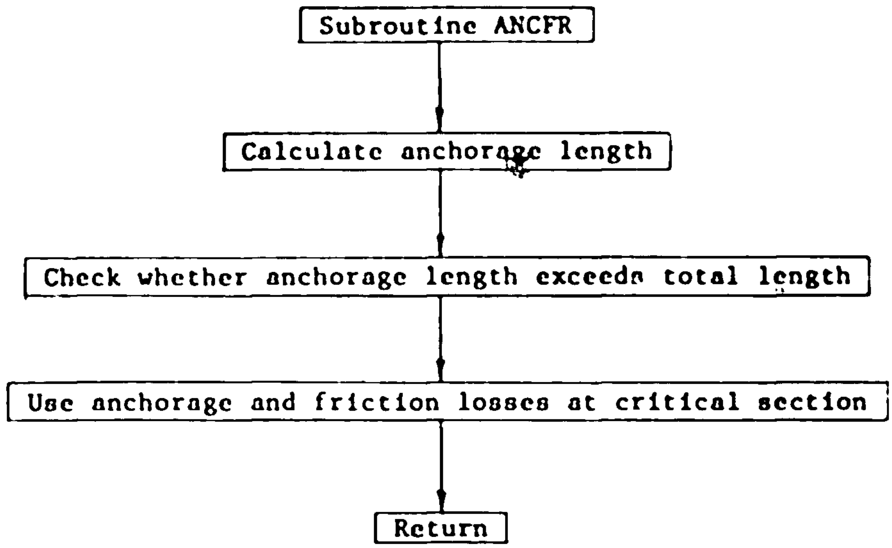
Flow Chart of Program FOURO2

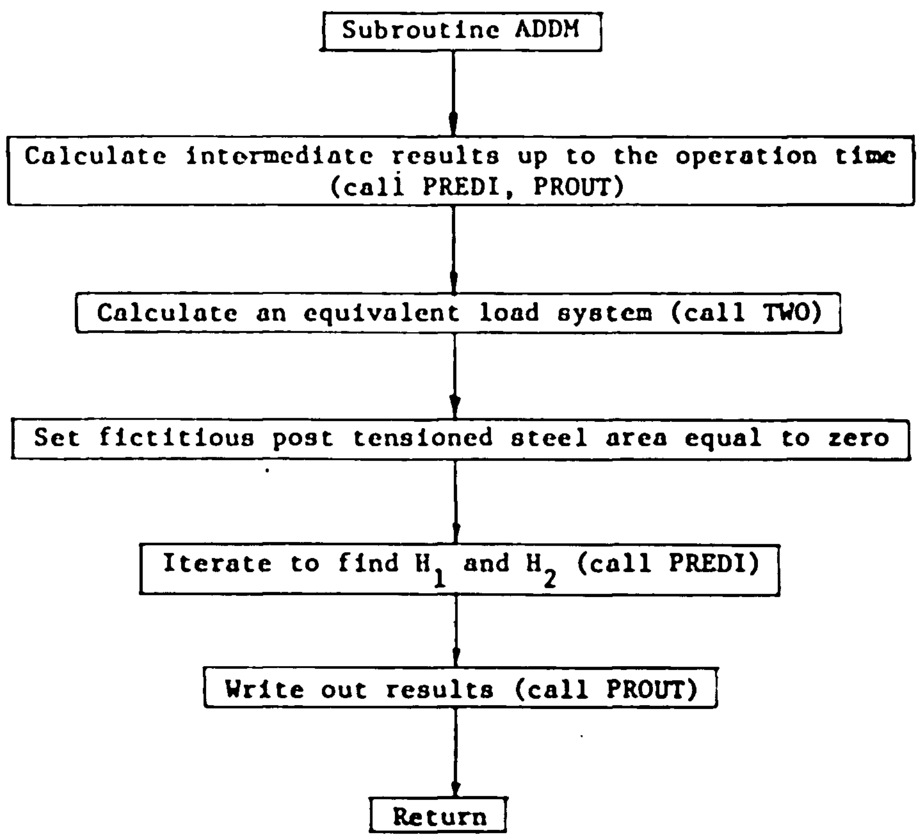


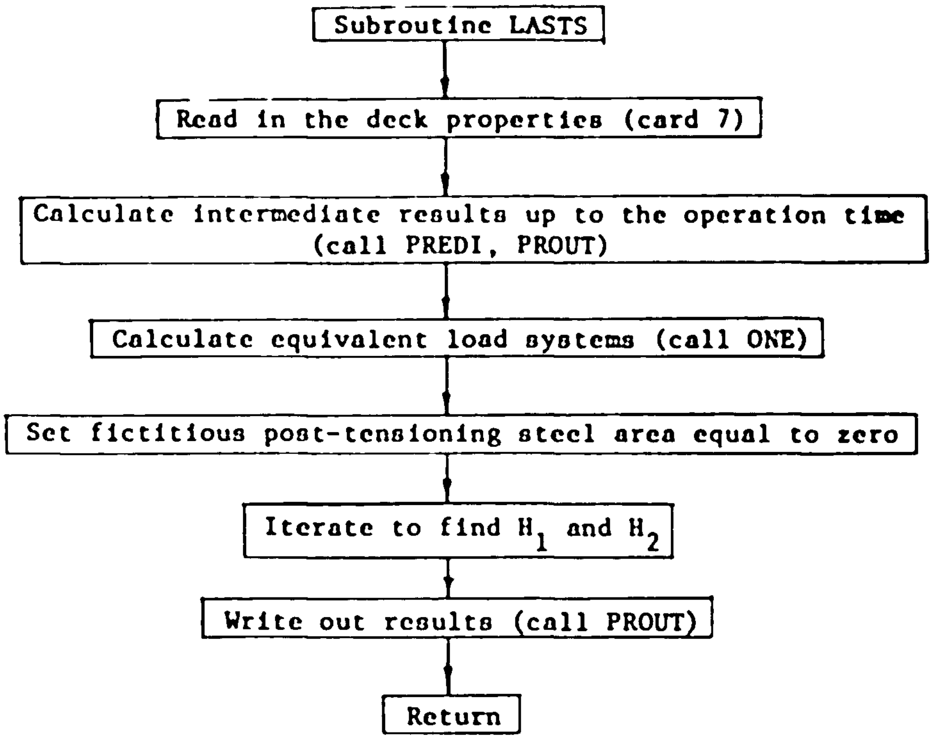

Subroutine INITI
Initially define the concrete coefficients
Calculate deadweight moment
Return

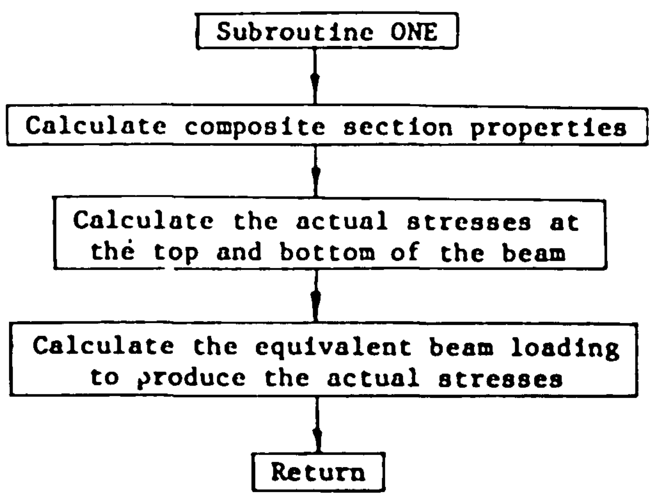

Subroutine PRE
Read in data for one layer of pretensioned strands (card 3)
Calculate steel stress and strain at tensioning
No
Have all layers of strands been tensioned?
Yes
Calculate steel stresses and strains prior to release
Calculate results immediately after release (call PREDI, PROUT)
Return

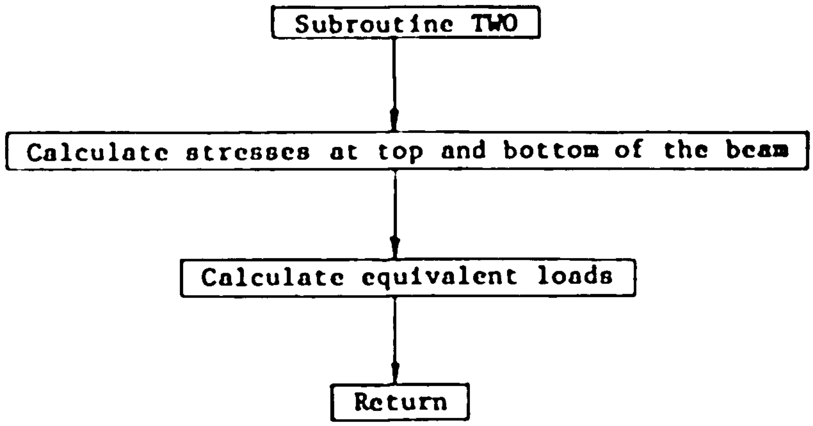

Subroutine POST
Read in second card for post-tensioning (card 5)
Calculate results immediately before tensioning (call PREDI, PROUT)
Calculate anchorage and friction losses (call ANCFR)
Calculate equivalent external loadings
Find H_1 and H_2 by iteration (call PREDI in each iteration)
Calculate steel and concrete stresses after post-tensioning (call PREDI)
Calculate k_{4i} for each strand
Calculate transformed section properties
Print out results (call PROUT)
Return

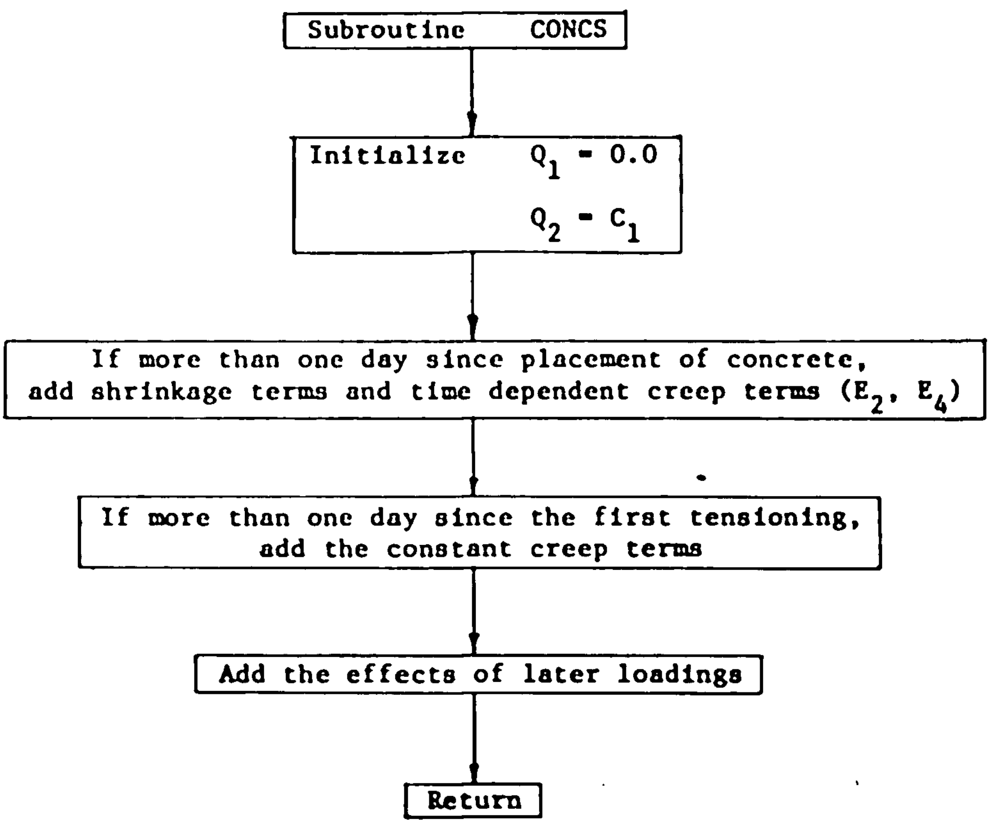

Subroutine ANCFR
Calculate anchorage length
Check whether anchorage length exceeds total length
Use anchorage and friction losses at critical section
Return

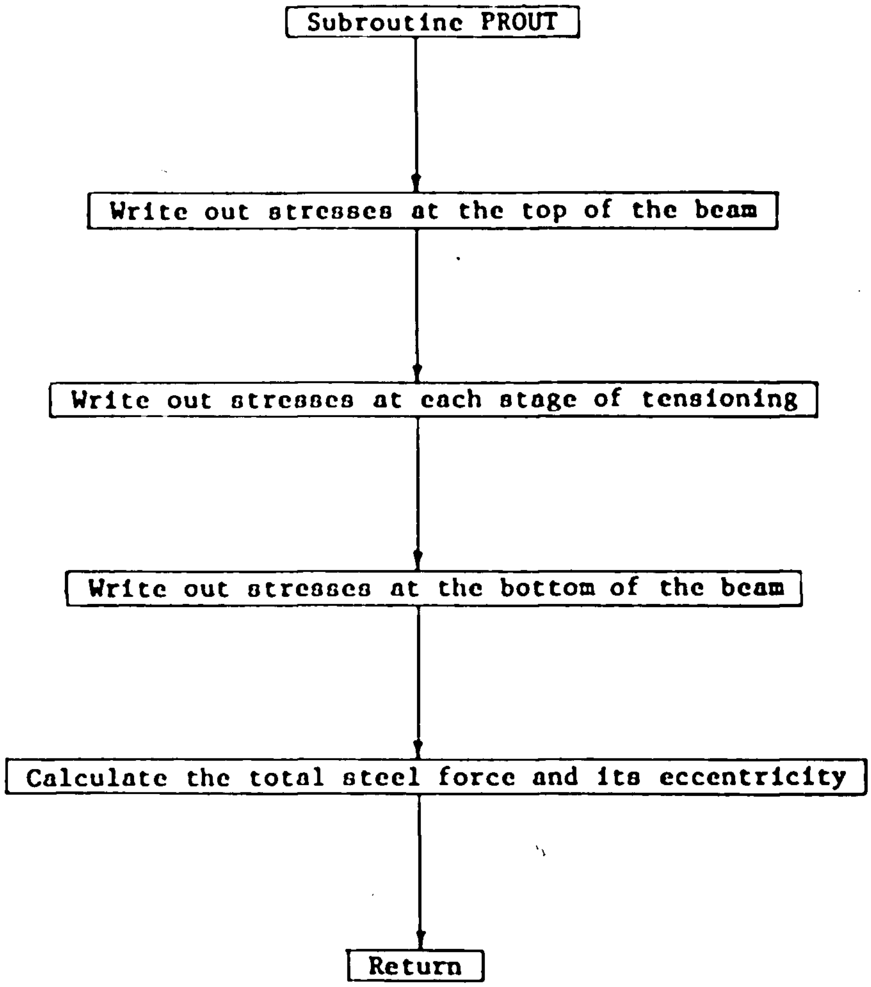

Subroutine ADDM
Calculate intermediate results up to the operation time (call PREDI, PROUT)
Calculate an equivalent load system (call TWO)
Set fictitious post tensioned steel area equal to zero
Iterate to find H_1 and H_2 (call PREDI)
Write out results (call PROUT)
Return


Subroutine LASTS
Read in the deck properties (card 7)
Calculate intermediate results up to the operation time (call PREDI, PROUT)
Calculate equivalent load systems (call ONE)
Set fictitious post-tensioning steel area equal to zero
Iterate to find H_1 and H_2
Write out results (call PROUT)
Return


Subroutine ONE
Calculate composite section properties
Calculate the actual stresses at the top and bottom of the beam
Calculate the equivalent beam loading to produce the actual stresses
Return


Subroutine TWO
Calculate stresses at top and bottom of the beam
Calculate equivalent loads
Return


Subroutine CONCS
Initialize $Q_1 = 0.0$
$Q_2 = C_1$
If more than one day since placement of concrete, add shrinkage terms and time dependent creep terms (E_2, E_4)
If more than one day since the first tensioning, add the constant creep terms
Add the effects of later loadings
Return


Subroutine PROUT
Write out stresses at the top of the beam
Write out stresses at each stage of tensioning
Write out stresses at the bottom of the beam
Calculate the total steel force and its eccentricity
Return

APPENDIX C

Input for Computer Program "FOURO2"

Format	Cols.	Symbol	Description
Card No. 1 (one card)			
F10.2	1-10	IPRE	Number of pretensioned levels
Card No. 2 (one card)			
F10.2	1-10	AGR	Net area of concrete in the beam (in^2)
F10.2	11-20	CMI	Moment of inertia of the net area of concrete (in^4)
F10.2	21-30	YNET	Distance from the top of the beam to the centroid of the net section (in.)
F10.2	31-40	SPANL	Span length (ft.)
F10.2	41-50	NSUCO	Stress-strain-time relationship for concrete of various manufacturers
F10.2	51-60	DEPTH	Depth of beam (in.)
Card No. 3 (one card for each level of pretensioning strand)			
F10.2	1-10	TIME(I)	Time of tensioning strand (negative number of days)
F10.2	11-20	NTYST	Type of strands
F10.2	21-30	NSUST	Stress-strain-time relationship for strands of various manufacturers
F10.2	31-40	STRANDS	Number of strands in the level

Format	Cols.	Symbol	Description
F10.2	41-50	YDIST(I)	Distance from top of beam to level of steel (in.)
F10.2	51-60	FSP(I)	Initial stress in strand (in fractions of guaranteed ultimate strength) in ksi/ksi

Card No. 4 (one card, alternating with card 5 for each post-tensioning)

F10.2	1-10	TIME(I)	Time of tensioning strand (days)
F10.2	11-20	NTYST	Type of strands
F10.2	21-30	NSUST	Stress-strain-time relationship for strands of various manufacturers
F10.2	31-40	STRANDS	Number of strands in the level
F10.2	41-50	YDIST(I)	Distance from top of beam to level of steel (in.)
F10.2	51-60	FSP(I)	Initial jacking stress (in fractions of guaranteed ultimate strength) in ksi/ksi

Card No. 5 (one card, immediately following card 4 for each post-tensioning)

F10.2	1-10	Y	Number of ends the strand is jacked from
F10.2	11-20	XKK	Wobble Coefficient
F10.2	21-30	DELT	Anchorage loss in in.
F10.2	31-40	XMU	Coefficient of friction
F10.2	41-50	EE	Steel depth at the end of the beam, in in.
F10.2	51-60	CMBM	Extra moment added to the beam in k-in.
F10.2	61-70	ALENGTH	New length of member, in ft. (if no change leave blank)

Format	Cols.	Symbol	Description
Card No. 6	One card, after last post-tensioning, before deck is placed a blank		
Card No. 7	(one card, immediately following card 6)		
F10.2	1-10	STIME	Time deck is added in days
F10.2	11-20	FCSL	Strength of deck in ksi
F10.2	21-30	FCBM	Strength of beam in ksi
F10.2	31-40	CMBM	Extra moment added to the beam in k-in.
F10.2	41-50	TSL	Structural thickness of deck in in.
F10.2	51-60	TSLW	Total thickness of the deck in in.
F10.2	61-70	WSL	Effective width of the deck in in.
F10.2	71-80	CMCO	Extra moment added to the composite section in k-in.
Card No. 8	(can be used anywhere after card No. 3)		
F10.2	1-10	STIME	Time extra moment is added in days
F10.2	11-20	CMCO	Extra moment added to the section in k-in.

Note: All time parameters are referenced to zero time at end of curing of concrete. Therefore, for pretensioned strands, the time of tensioning (TIME(I) on card 3) has a negative value.

10. VITA

Burt Hoffman was born November 16, 1953 in Abington, Pennsylvania, the son of Alvin and Gail Hoffman. He was educated in the Springfield Township, Montgomery County, Pennsylvania public school system. In September 1971 he enrolled at Lehigh University where he received his Bachelor's Degree in the Department of Civil Engineering in June 1975.

Following graduation he joined Spotts, Stevens, and McCoy, Inc., as a structural engineer. In June 1976 he returned to Lehigh University as a graduate student and research assistant in the Fritz Engineering Laboratory. He was associated with the research project Prestress Loss in Post-Tensioned Members in the Structural Concrete Division.