DIRECT METHOD FOR ESTIMATING PRESTRESS LOSSES

Ti Huang, Lehigh University

A rational method for estimating prestress losses in pretensioned concrete members is presented. The method is based on linking experimentally developed stress-strain-time relationships of steel and concrete materials. It enables a direct determination of stress and strain distributions in a member at any time within the service life of the member and avoids the need for step-by-step methods. Wide ranges of variation for concrete material characteristics and other design factors are permitted. The new method is illustrated by a practical design problem. Comparisons with 2 recently proposed procedures show good agreement of results.

•METHODS for estimating prestress losses in prestressed concrete members vary widely. At one time, a simple flat percentage or flat value was used in many design codes (1, 2). Today, complicated procedures involving the use of numerous equations, tables, charts, and a step-by-step method of calculation are required (3, 4). Neither extreme is satisfactory to the design engineers who want a simple, accurate, and flexible method that can accommodate variations in the numerous design and fabrication factors of prestressed concrete members.

This paper presents a rational method that permits a direct and accurate prediction of prestress losses throughout the service life of the member. Variations in material properties, geometrical design, and fabrication schedule are allowed for, and step-by-step calculations are avoided. In the present form, this new method applies only to pretensioned members, but the basic concept is equally valid for post-tensioned members.

In this paper, prestress is the stress in steel or concrete when all external loads, including the weight of the member, are temporarily and instantaneously removed. Consequently, the actual stress under a loaded condition is the sum of prestress and stress caused by all prevailing loads. Loss of prestress is due to the initial steel stress after anchorage. Therefore, prestress loss for a pretensioned member includes the effects of elastic shortening, shrinkage, creep, and relaxation. But friction and anchorage losses are not considered.

BASIC CONCEPT

The basic concept of the new method involves the use of stress-strain-time relationships to represent elastic as well as long-term rheological behavior of the steel and concrete materials. In the most general form, these relationships are given by the following equations:

$$f_s = f(\epsilon_s, t_s) = f_{s,e\ell} - f_{s,re\ell} \quad (1)$$

$$\epsilon_c = g(f_c, t_c) = \epsilon_{c,e\ell} + \epsilon_{c,sh} + \epsilon_{c,cr} \quad (2)$$

Equation 1 shows the steel tensile stress f_s as a function of steel strain ϵ_s and time after tensioning t_s, and as the difference between the elastic stress $f_{s,e\ell}$ and relaxation

---

Publication of this paper sponsored by Committee on Concrete Bridges.

loss $f_{s,re\ell}$. Equation 2 shows the concrete compressive strain ϵ_c as a function of concrete fiber stress f_c and time after end of curing t_c. It is the sum of elastic, shrinkage, and creep components. Here it is assumed that transfer of prestress occurs immediately after curing. Hence shrinkage and creep are controlled by the same time factor.

For a pretensioned concrete member, the stress-strain-time relationships of the concrete and steel materials are linked by the following 3 sets of linking conditions:

1. Time compatibility,

$$t_s - t_c = k_1 \quad (3)$$

2. Strain compatibility at the location of each prestressing strand,

$$\epsilon_s + \epsilon_c = k_2 \quad (4)$$

3. Equilibrium conditions over the cross section,

$$\int f_c dA_c - \Sigma f_s a_{ps} = P \quad (5)$$

$$\int f_c x dA_c - \Sigma f_s x a_{ps} = -M \quad (6)$$

where

k_1 = time interval from tensioning of steel to transfer of prestress (this includes time for form setting, casting, and curing),
k_2 = initial tensioning strain in steel,
A_c = area of net concrete section,
a_{ps} = area of individual prestressing element,
x = distance to elementary area from the centroidal horizontal axis,
P = thrust on section caused by external loads, and
M = bending moment on section caused by external loads.

The positive directions of x, P, and M are shown in Figure 1. In equations 5 and 6 the integrations are over the entire net concrete area, and the summations are over all pretensioning elements. All of the quantities defined for equations 3 to 6 are design or fabrication factors and are known or specified for the estimation of prestress losses. Thus equations 1 through 6 represent a set of 6 conditions for the 2 time variables t_s and t_c and the 4 stress and strain variables ϵ_s, ϵ_c, f_s, and f_c, which are functions of the location parameter x. A reasonable assumption was made that concrete stress varies linearly across the section

$$f_c = g_1 + g_2 x \quad (7)$$

When this condition is added, sufficient equations are available to evaluate all unknowns for any given time. That is, the time variations of stresses and strains can be determined. Thus, when the member design and the initial conditions k_1 and k_2 are known, a complete solution of the stress and strain distribution can be obtained by repeatedly solving equations 1 through 7 for different values of time. It is important to note that, for any specified time, solution is direct and not dependent on the solution at preceding times. Thus step-by-step accumulation is not needed. It also should by pointed out that f_s and f_c in the aforementioned equations include the effects of applied loads and, therefore, are not the prestresses as defined earlier. By definition, steel prestress and prestress loss are evaluated by the following equations:

$$f_p = f_s - f_{s\ell} \quad (8)$$

$$\Delta f_p = f_{si} - f_p = f_{si} - f_s + f_{s\ell} \quad (9)$$

where

f_p = steel prestress,
$f_{s\ell}$ = steel stress caused by applied loads including member weight and all permanent loads,
Δf_p = loss of prestress, and
f_{si} = initial steel stress immediately at anchorage.

STRESS-STRAIN-TIME RELATIONSHIPS

The functions f and g in equations 1 and 2 were developed experimentally based on observations of elastic, relaxation, creep, and shrinkage behavior of simple steel and concrete specimens. Steel relaxation data were obtained from strand specimens tested in fixed-length loading frames under various initial tensile stresses. To gather information on concrete strains, we used concentrically pretensioned rectangular concrete specimens in conjunction with similar specimens containing untensioned strands.

In selecting time functions for regression analyses of relaxation, shrinkage, and creep data, we placed special emphasis on the suitability of these functions for extrapolation because long-term projections based on short-term observations would be necessary. For this purpose, we made analyses by using data covering different periods of time and compared them with projected values for an arbitrarily chosen future time (100 years after tensioning). Lack of sensitivity of the projected final value to the amount of experimental data used in the analysis was used as a criterion in selecting time functions (5). A modified form of the logarithmic function was chosen because it was simple and because it satisfied the criterion of insensitivity.

Relaxation loss data from steel strand specimens were first analyzed for time and initial stress. The resulting expression then was combined with the elastic stress-strain relationship to form the stress-strain-time equation. The form of this equation is as follows:

$$f_s = f_{pu}\{A_1 + A_2\epsilon_s + A_3\epsilon_s^2 - [B_1 + B_2 \log (t_s + 1)]\epsilon_s - [B_3 + B_4 \log (t_s + 1)]\epsilon_s^2\} \quad (10)$$

where f_{pu} = specified ultimate tensile strength of steel in kips per square inch (megapascals). f_s is measured in kips per square inch (megapascals); ϵ_s is measured in units $\times 10^{-2}$; t_s is measured in days starting from initial tensioning.

The applicability of equation 10 is restricted because of the limited test ranges of the controlled factors. These ranges are

$$0.5 \leq f_s/f_{pu} \leq 0.8$$

$$1 \leq t_s \leq 36{,}500$$

The experimental work dealt with 270-kips/in.^{2}-grade (1860-MPa), stress-relieved, 7-wire strand specimens $^7/_{16}$ in. (1.11 cm) and $^1/_2$ in. (1.27 cm) in size. No significant size effect was found. The values of the regression coefficients are given in Table 1.

Equation 2 for concrete characteristics was developed in a similar manner by combining expressions representing elastic, shrinkage, and creep strains (6). Elastic strains were measure directly at the time of prestress transfer. The shrinkage strain of a prestressed member was defined to be the same as that of plain concrete containing no reinforcement. Creep strain was obtained from the measured total strain by deducting elastic-shrinkage and elastic-rebound strains. Time function for shrinkage and creep strains was selected by using the same criteria that were used for relaxation behavior. Coincidentally, the same function was chosen. The functional form of the concrete stress-strain relationship is as follows:

Figure 1. Sign convention for applied loads.

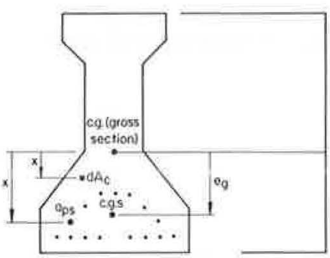


Table 1. Coefficients for steel stress-strain-time relationship.

Coefficient	Value	Coefficient	Value
Elastic		Relaxation	
A_1	-0.04229	B_1	-0.05867
A_2	1.21952	B_2	0.00023
A_3	-0.17827	B_3	0.11860
		B_4	0.04858

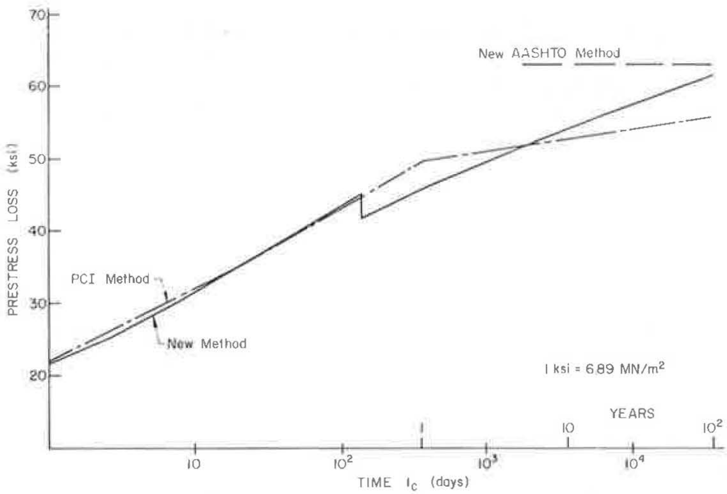
Note: All coefficients are dimensionless and are the same in SI units.

Table 2. Coefficients for concrete stress-strain-time relationship.

Coefficient	Value: Upper Bound Loss	Value: Lower Bound Loss
Elastic strain, C_1	0.02500	0.02105
Shrinkage		
D_1	-0.00668	-0.00066
D_2	0.02454	0.01500
Creep		
E_1	-0.01280	-0.00664
E_2	0.00675	-0.00331
E_3	-0.00060	-0.00371
E_4	0.01609	0.01409

Note: C_1, E_3, and E_4 values will be combined with f_c. Multiply C_1, E_3, and E_4 values by 0.145 to convert to megapascals.

Figure 2. Example problem for predicted prestress losses.



$$\epsilon_c = C_1 f_c + [D_1 + D_2 \log (t_c + 1)] + [E_1 + E_2 \log (t_c + 1)] + f_c [E_3 + E_4 \log (t_c + 1)] \quad (11)$$

ϵ_c is measured in units $\times 10^{-2}$; f_c is measured in kips per square inch (megapascals); and t_c is measured in days starting from the time of transfer, which is the same as the end of curing period.

In the experimental study, 2 concrete mixes were used, both of which satisfy the same minimum strength requirements [5.0 kips/in.^{2} (34.5 MPa) at transfer and 5.5 kips/in.^{2} (37.9 MPa) at 28 days]. However, their composition and manufacturing procedure were sufficiently different so that their rheological behaviors differed significantly. Two sets of regression coefficients were developed to reflect this wide variation. They are given in Table 2. The applicability of equation 11 is restricted because of the limited test ranges of the controlled factors. These ranges are

$0 \le f_c \le 3.3$ kips/in.^{2} (22.8 MPa)

$1 \le t_c \le 36{,}500$

FORMULATION OF PROCEDURE

For any specified time, equation 10 reduces to a simple quadratic function of ϵ_s. Equation 11 is linear in terms of f_c. Their combination with equations 3 through 7 results in a pair of simultaneous quadratic equations in g_1 and g_2. The solution of g_1 and g_2 then enables one to evaluate steel and concrete stresses and strains over the entire cross section. Note that a general solution in this manner would result in different losses in the several prestressing elements and thus cause a gradual shift of the centroid of prestressing.

For practical purposes, all prestressing steel usually is regarded as concentrated at 1 point, the c.g.s., for stress calculations. When this simplification is used, the simultaneous quadratic equations can be simplified into a single quadratic equation in terms of the concrete fiber stress at c.g.s., f_{cs}, as follows:

$$(R_1 - \beta f'_{c\ell}) + (R_2 + 1 - \beta) f_{cs} + R_3 f_{cs} = 0 \quad (12)$$

where

f_{cs} = concrete fiber stress at c.g.s. in kips per square inch (megapascals),
= $g_1 + g_2 e_g$,
$f'_{c\ell}$ = nominal concrete fiber stress at c.g.s. caused by applied loads in kips per square inch (megapascals),
= $-\frac{P}{A_g} + \frac{M e_g}{I_g}$ (tension positive), and
β = dimensionless geometrical parameter.

β also can be represented as follows:

$$\beta = \frac{1}{A_{ps}\left(\frac{1}{A_g} + \frac{e_g^2}{I_g}\right)} = \frac{A_g I_g}{A_{ps}(I_g + A_g e_g^2)}$$

where

A_g = area of gross cross section in square inches (square centimeters),

I_g = moment of inertia of gross cross section in inches^4 (centimeters^4),
e_g = eccentricity of prestress for gross cross section in inches (centimeters), and
A_{ps} = total area of prestressing steel in square inches (square centimeters).

Equilibrium equations 5 and 6 also can be simplified to yield the value of steel stress at any arbitrary time:

$$f_s = (\beta - 1)f_{cs} + \beta f'_{c\ell} \quad (13)$$

DERIVATIONS OF EQUATIONS

The set of equations used in the development of the basic analytical procedure includes the 2 stress-strain-time relationships (equations 10 and 11), the 4 linking relationships (equations 3 through 6), and the linear relationship defining concrete stress distribution in the member section (equation 7). In these equations, f_c, f_s, ϵ_c, and ϵ_s are functions of x. In equations 5 and 6, the integrations are over the net concrete section area, and the summations cover all prestressing steel elements. Substituting equation 7 into 5 and 6 and performing the integrations yields

$$A_g g_1 - \Sigma(f_s + f_{cs})a_{ps} = P \quad (14)$$

$$I_g g_2 - \Sigma(f_s + f_{cs})x_s a_{ps} = -M \quad (15)$$

where

f_{cs} = concrete fiber stress at the level of prestress steel, and
x_s = x distance for an individual prestressing element.

Therefore,

$$f_{cs} = g_1 + g_2 x_s \quad (16)$$

To simplify further derivation, we introduce a group of parameters.

$$P_1 = A_1 f_{pu}$$

$$P_2 = [A_2 - B_1 - B_2 \log(t_s + 1)]f_{pu}$$

$$P_3 = [A_3 - B_3 - B_4 \log(t_s + 1)]f_{pu}$$

$$Q_1 = D_1 + E_1 + (D_2 + E_2) \log(t_c + 1)$$

$$Q_2 = C_1 + E_3 + E_4 \log(t_c + 1)$$

Then,

$$f_s = P_1 + P_2\epsilon_s + P_3\epsilon_s^2 \quad (17)$$

$$\epsilon_c = Q_1 + Q_2 f_c \quad (18)$$

Substituting this information into equation 4 yields

$$\epsilon_s = k_2 - Q_1 - Q_2 f_{cs} \quad (19)$$

Substituting this into equation 16 gives

$$f_s = P_1 + P_2(k_2 - Q_1 - Q_2 f_{cs}) + P_3(k_2 - Q_1 - Q_2 f_{cs})^2$$

$$= R_1 + R_2 f_{cs} + R_3 f_{cs}^2 \qquad (20)$$

where

$$R_1 = P_1 + P_2(k_2 - Q_1) + P_3(k_2 - Q_1)^2,$$

$$R_2 = - Q_2[P_2 + 2P_3(k_2 - Q_1)], \text{ and}$$

$$R_3 = P_3 Q_2^2.$$

Substituting equations 16 and 20 into equilibrium condition equations 14 and 15 gives

$$A_g g_1 - \Sigma[R_1 + (R_2 + 1)(g_1 + g_2 x_s) + R_3(g_1 + g_2 x_s)^2] a_{ps} = P \qquad (21)$$

$$I_g g_2 - \Sigma[R_1 + (R_2 + 1)(g_1 + g_2 x_s) + R_3(g_1 + g_2 x_s)] x_s a_{ps} = -M \qquad (22)$$

These equations are simultaneous quadratic equations in g_1 and g_2.

In the simplified case when prestressing steel is regarded as concentrated at 1 level, x_s becomes a constant for all elements and is equal to e_g by definition.

Replacing x_s by e_g in equations 21, 22, and 19 gives

$$A_g g_1 - [R_1 + (R_2 + 1)(g_1 + g_2 e_g) + R_3(g_1 + g_2 e_g)^2] A_{ps} = P \qquad (23)$$

$$I_g g_2 - [R_1 + (R_2 + 1)(g_1 + g_2 e_g) + R_3(g_1 + g_2 e_g)^2] A_{ps} e_g = -M \qquad (24)$$

$$f_{cs} = g_1 + g_2 e_g \qquad (25)$$

Multiply equation 23 by I_g and multiply equation 24 by $A_g e_g$; add these 2 equations and substitute equation 25. This gives

$$A_g I_g f_{cs} - [R_1 + (R_2 + 1) f_{cs} + R_3 f_{cs}^2] A_{ps}(I_g + A_g e_g^2) = PI_g - MA_g e_g$$

Therefore,

$$f_{cs} - [R_1 + (R_2 + 1) f_{cs}] A_{ps} \left(\frac{1}{A_g} + \frac{e_g^2}{I_g}\right) = \frac{P}{A_g} - \frac{Me_g}{I_g} \qquad (26)$$

When we introduce

$$\beta = \frac{1}{A_{ps}\left(\frac{1}{A_g} + \frac{e_g^2}{I_g}\right)}$$

$$f'_{c\ell} = -\frac{P}{A_g} + \frac{Me_g}{I_g}$$

equation 26 is transformed into equation 12.

It is important to note that $f'_{c\ell}$ is the nominal concrete stress caused by the applied loads based on gross section properties. It uses a tension positive sign. The dimensionless geometrical factor β is associated closely with the ratio of steel prestress to concrete prestress.

Equation 13 for steel stress is obtained by subtracting equation 12 from equation 20.

ANALYSIS PROCEDURE

The procedure for an analysis of prestress losses in a pretensioned member is as follows when material, geometric, and fabrication factors, including the concrete characteristics β, f_c', k_1, and k_2, are known or specified for the problem:

1. Evaluate R_1, R_2, and R_3 for each specified time t_c;
2. Solve equation 12 for f_{cs};
3. Evaluate the steel stress f_s by using equation 13;
4. Calculate concrete and steel strains ϵ_c and ϵ_s by using equations 2 and 4 respectively; and
5. Evaluate steel prestress and prestress loss by using equations 8 and 9 respectively.

EXAMPLE AND COMPARISON

This new method enables a direct solution of prestress loss at any time during the service life of the member without requiring a step-by-step accumulative technique. However, to determine the complete history of prestress variation in a member, the analysis procedure must be repeated many times for different values of t_c. The number of calculations involved is considerable. A computer program has been developed to carry out these calculations.

An example is presented here to illustrate calculations according to the new procedure and to compare the results with those from other procedures. This example deals with a Pennsylvania Department of Transportation standard 20/33 I-beam (2) that spans 60 ft (18.3 m) center to center. This beam is prestressed with thirty-four ½-in. (1.27-cm) stress-relieved strands of the 270-kips/in.^{2} (1860-MPa) grade. The concrete used corresponds to the lower bound of prestress losses. e_g = 7.95 in. (20.2 cm); k_1 = 2.3 days; and f_{si} = 183.6 kips/in.^{2} (1266 MPa) = 0.68 f_{pu}. The beam is part of a highway bridge on which the deck slab is 7.5 in. (19.05 cm) thick and cast in place 140 days after transfer. The spacing between beams is 6 ft 10 in. (208 cm) center to center. An additional dead load of 30 lb/ft^2 (1440 N/m^2) is applied later to be resisted by the composite section.

For the sake of simplicity, the 30-lb/ft^2 (1440-N/m^2) superimposed load is treated as applied together with deck gravity load at 140 days. From the geometry of the given section, it is calculated that β = 50.5. Before application of superimposed loads, $f_{c\ell}'$ = 0.417 kips/in.^{2} (2.88 MPa) and $f_{s\ell}$ = 1.93 kips/in.^{2} (13.3 MPa). Afterwards, $f_{c\ell}'$ = 1.171 kips/in.^{2} (8.07 MPa) and $f_{s\ell}$ = 5.4 kips/in.^{2} (37.4 MPa).

Detailed calculations according to the new procedure are illustrated for the time just before the application of deck and other superimposed loads. At that time, t_c = 140 days, t_s = 142.3 days, $f_{c\ell}'$ = 0.417 kips/in.^{2} (2.88 MPa), and $f_{s\ell}$ = 1.93 kips/in.^{2} (13.3 MPa). The coefficients R_1, R_2, and R_3 in equation 12 will now be evaluated.

From the steel stress-strain relationship (coefficients A_1, A_2, and A_3 from Table 1) for the initial tensioning stress, f_{si} = 0.68 f_{pu} and k_2 = 0.65509.

$$P_1 = -0.04229\ (270) = 11.4$$

$$P_2 = [1.21952 - (-0.05867) - 0.00023 \log (142.3 + 1)](270) = 345.0$$

$$P_3 = [-0.17827 - 0.11860 - 0.04858 \log (142.3 + 1)](270) = -108.4$$

$$Q_1 = -0.00066 - 0.00664 + (0.01500 - 0.00331) \log (140 + 1) = 0.0178$$

$$Q_2 = 0.02105 - 0.00371 + 0.01409 \log (140 + 1) = 0.0476$$

$$k_2 - Q_1 = 0.655 - 0.0178 = 0.637$$

$$R_1 = -11.4 + 345.0\ (0.637) - 108.4\ (0.637)^2 = 164.4$$

$$R_2 = -0.0476\,[345.0 - 2(108.4)(0.637)] = -9.85$$

$$R_3 = -108.4(0.0476)^2 = -0.246$$

Substituting this into equation 12 gives

$$(164.4 - 50.5 \times 0.417) + (-9.85 - 49.5)f_{os} - 0.246\, f_{os}^2 = 0$$

or, more simply

$$143.3 - 59.3\, f_{os} - 0.246\, f_{os}^2 = 0$$

The solution for f_{os} is 2.39 kips/in.^{2} (16.5 MPa). From equation 13

$$f_s = 49.5\,(2.39) + 50.5\,(0.417) = 139.5 \text{ kips/in.}^2 \text{ (962.5 MPa)}$$

Hence

$$f_p = 139.5 - 1.93 = 137.6 \text{ kips/in.}^2 \text{ (959.4 MPa)}$$

$$\Delta f_p = 183.6 - 137.6 = 46.0 \text{ kips/in.}^2 \text{ (317 MPa)}$$

It should be reemphasized that prestress loss is calculated directly from initial and present conditions without any reference to intervening loading history. Figure 2 shows the computer results of similar calculations at other times.

From Figure 2, it is easily seen that the growth of prestress loss is nearly linear with respect to log t_o as long as the load remains unchanged. It would be reasonable, therefore, to simplify the calculating procedure by taking advantage of this phenomenon. Direct solution will be needed only at a few key stages, and prestress loss at an intermediate time can be estimated easily by means of this linear semilogarithmic relationship.

Figure 2 also shows estimates based on a step-by-step procedure recommended by the Prestressed Concrete Institute (PCI) (4) and by a procedure of the American Association of State Highway and Transportation Offlcals (AASHTO) (8). Calculation according to a 1973 specification (7) resulted in an extremely high loss estimate of nearly 80 kips/in.^{2} (552 MPa) and was not shown in Figure 2. Two of these methods appear to have implicitly defined prestress to include the stress caused by applied loads (4, 8). For the comparison to be meaningful, all estimates shown in Figure 2 have been adjusted to conform to the definition for prestress given in this paper.

Very good agreement is noted between the PCI method (4) and the new method presented in this paper, particularly during the initial period before the increase of external load. The low estimate of the final loss by the PCI method [55.5 kips/in.^{2} (383 MPa)] is believed to be a reflection of a relatively short assumed service life.

The AASHTO method (8) deals with the final loss only and does not yield as much information as the other 2 methods do. Although the final loss predicted by the AASHTO method [62.6 kips/in.^{2} (432 MPa)] appears to agree quite well with the prediction by the new method [61.1 kips/in.^{2} (421 MPa)], there are indications that AASHTO also considered a service life shorter than 100 years. Consequently, it would be more appropriate to recognize the difference and conclude that the AASHTO method results in slightly higher loss predictions than the new method does. It should be reiterated that, in this example, concrete corresponding to lower bound losses is considered. The new method is very sensitive to the characteristics of concrete, but the AASHTO method is not. In the AASHTO method, only the elastic loss is affected. When the same example was repeated using high-loss concrete, the new method yielded a final loss of 76.9 kips/in.^{2} (530 MPa) at 100 years; the AASHTO method resulted in a significantly lower loss of only 65.4 kips/in.^{2} (451 MPa) at an unspecified time. Similar comparisons have been observed in other examples. In general, it can be stated that the AASHTO procedure yields predicted final loss values lying within the range predicted by the new method, but they are much closer to the lower bound.

CONCLUSION

The new method for estimating prestress losses is a workable alternative to the several methods currently available. It allows for wide ranges of variation in material characteristics and other design factors, and it enables the direct determination of prestress loss at any time during the service life of the member.

ACKNOWLEDGMENT

The work presented in this paper is part of a research project recently concluded at Lehigh University (9). The 7-year project was supported financially by the Pennsylvania Department of Transportation, the Federal Highway Administration, and the Reinforced Concrete Research Council.

REFERENCES

1. Standard Specifications for Highway Bridges. American Association of State Highway Officials, 10th Ed., 1969.
2. Standards for Prestressed Concrete Bridges. Pennsylvania Department of Transportation, Forms ST202 to ST208, 1964.
3. International Recommendations for the Design and Construction of Concrete Structures. European Commission on Concrete, 1970.
4. Tentative Recommendation for Estimating Prestress Losses. Committee on Prestress Losses, Prestressed Concrete Institute, final draft, Sept. 1973.
5. E. G. Schultchen, H. T. Ying, and T. Huang. Relaxation Behavior of Prestressing Strands. Lehigh Univ., Fritz Laboratory Rept. 339.6, March 1972.
6. H. T. Ying, E. G. Schultchen, and T. Huang. Estimation of Concrete Strains and Prestress Losses in Pretensioned Members. Lehigh Univ., Fritz Laboratory Rept. 339.7, May 1972.
7. Standard Specification for Highway Bridges. American Association of State Highway Officials, 11th Ed., 1973.
8. Proposed Revision to Standard Specification for Highway Bridges, Article 1.6.7. Committee on Structures and Bridges, American Association of State Highway and Transportation Officials, 1974.
9. T. Huang. Prestress Losses in Pretensioned Concrete Structural Members. Lehigh Univ., Fritz Laboratory Rept. 339.9, Aug. 1973.