NEW LOSS ESTIMATION FOR DESIGN OF POST-TENSIONED SPLICED GIRDER BRIDGES

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ABSTRACT

Spliced girder bridges are growing in popularity due to their high quality, and efficient and cost-effective construction, especially for larger spans. Additionally, complex time-dependent analysis, which has historically presented challenges to engineers, is now quite feasible with the availability of new tools.

Formulas currently used by various codes are based on concrete strengths of up to 6.0 ksi[1]. NCHRP project 18-07[2] has resulted in new methods and equations for calculation of loss of prestress. The project targeted high-strength concrete, although its results apply to the entire range of concrete strength in current practice. Although the NCHRP 18-07 procedures are more accurate than current code equations, they have been calibrated to prestressed girders and their effect on the post-tensioned tendons should be evaluated.

This paper presents the results of a study of the effects of the new equations on spliced post-tensioned girder bridge structures. A bridge superstructure is analyzed with CONSPLICE®[3] using existing ACI[4], LRFD[5], CEB-FIP[6] model codes, and NCHRP 18-07[2] equations.

Keywords: Bridge, Prestress, Spliced Girder, Post-tensions, Prestress Loss, High-strength Concrete, Time-Dependent, Analysis, Shrinkage, Creep

INTRODUCTION

The primary purpose of calculating losses in prestressing is to accurately predict the stress and deformation in a concrete member under service conditions. In 2002, NCHRP project 18-07[2] resulted in new design guidelines for estimating prestress losses in pretensioned high-strength concrete bridge girders. In this project, a more accurate equation for calculating the modulus of elasticity of concrete was developed. A proposed formula for estimating shrinkage and creep was developed as well based on extensive test data. Calculation of relaxation in low-relaxation strands was simplified to a single loss value. The methods for calculating prestress loss developed in NCHRP 18-07[2] have proven to reasonably conservative and accurate in pretensioned high-strength concrete bridge girders. The goal of this research was to extend the existing LRFD methods to high strength concrete and not to dispute or modify them for normal strength concrete. The goal of this paper is to further this research by generalizing and adapting this new method of predicting prestress losses to spliced girder applications, which occupy a significant portion of the bridge market.

LOSS OF PRESTRESS

Prestress loss is the loss of compressive force acting on the concrete component of a prestressed concrete section. Prestress loss in pretensioned girders is the summation of the elastic losses and gains, shrinkage, creep and relaxation losses. Formulas currently used by various codes to determine concrete modulus of elasticity, shrinkage and creep have been empirically established based primarily on data for normal-strength concrete with compressive strength up to 6.0 ksi[1].

NCHRP project 18-07[2] dealt primarily with time-dependent loss prediction in high-strength pretensioned girders. It involved testing conducted using concrete mixes from four different states—Nebraska, New Hampshire, Texas, and Washington. Three HSC mixes and one deck concrete mix were tested from each state—a total of 16 mixes. Modulus of elasticity, strength, shrinkage, and creep specimens were tested. Components of prestress loss unaffected by concrete strength, i.e. post-tensioning losses such as friction and wobble, are not addressed in 18-07[2].

ELASTIC EFFECTS

Elastic effects are gains or losses that occur when an applied load causes a shortening or lengthening of the prestressing steel such that the force exerted on the concrete member by the steel changes. This results in the shortening or lengthening of the concrete member and an accompanying elastic gain or loss in the steel. Elastic effects affect concrete stress throughout the life of a prestressed element. In a spliced girder bridge with two-stage post-tensioning, elastic effects are introduced during the following events:

1) At transfer, elastic effects consist of a loss due to pretensioning force and a gain due to member self weight.

2) During first stage post-tensioning, they result in a loss in the pretensioning steel.

3) During deck placement, they consist of gains in both the pretensioning and first stage post-tensioning steel.

4) During second stage post-tensioning, they include losses in both pretensioning and first stage post-tensioning steel, assuming the first stage post-tensioning ducts have been grouted as is normally the case.

5) Finally, elastic effects at the application of superimposed dead loads and live loads consist of gains in pretensioning steel and both stages of post-tensioning steel, once again, assuming grouting.

Elastic effects are different in high-strength concrete as compared with normal strength concrete due to HSC's higher modulus of elasticity. The modulus of elasticity, E, of a material is defined as the change of stress with respect to strain in the linear elastic range—a material's stiffness. Concrete is an elastoplastic material and, therefore, has a modulus of elasticity (E_c) that varies non-linearly with stress. It is approximated as a linear value by drawing a line on the stress-strain curve from the origin to the point on the curve located at $f'_c/2$, i.e., the secant modulus at $f'_c/2$. Accurately approximating the value of the concrete modulus of elasticity allows for prediction of a component's initial camber and elastic prestress gains and losses. Variables influencing E_c include cement paste stiffness, porosity, and composition of the boundary zone between paste and aggregates, aggregate stiffness, and concrete constituent proportions.

NCHRP 18-07[2] included E_c laboratory tests of a set of three 4″ x 8″cylinders for each state, three HSC mixes, and each state's deck mix, a total of 16 mixes, at 1, 3, 7, 14, 28, 56, 90, 128 and 256 days. A total of 108 cylinders were tested in the laboratory. E_c field testing included 18 cylinders from each plant composed of mix selected for each state's instrumented girders. They were cured under the same conditions as the girders. The resulting equation includes the effect of aggregate type on E_c as well as the variability of concrete density with varying strength and includes a factor to represent the difference between national and local average E_c values.

The following are the NCHRP 18-07[2] proposed modulus of elasticity equations:

$$E_c = 33{,}000 K_1 K_2 w_c^{1.5} \sqrt{f'_c} \tag{1}$$

$$w_c = 0.140 + f'_c/1000 \tag{2}$$

K_1 = factor accounting for difference between national and local average E_c values

K_2 = factor allowing an upper or lower bound calculation

Fig. 1 shows NCHRP 18-07[2] modulus of elasticity test results and compares current prediction methods with the proposed method.

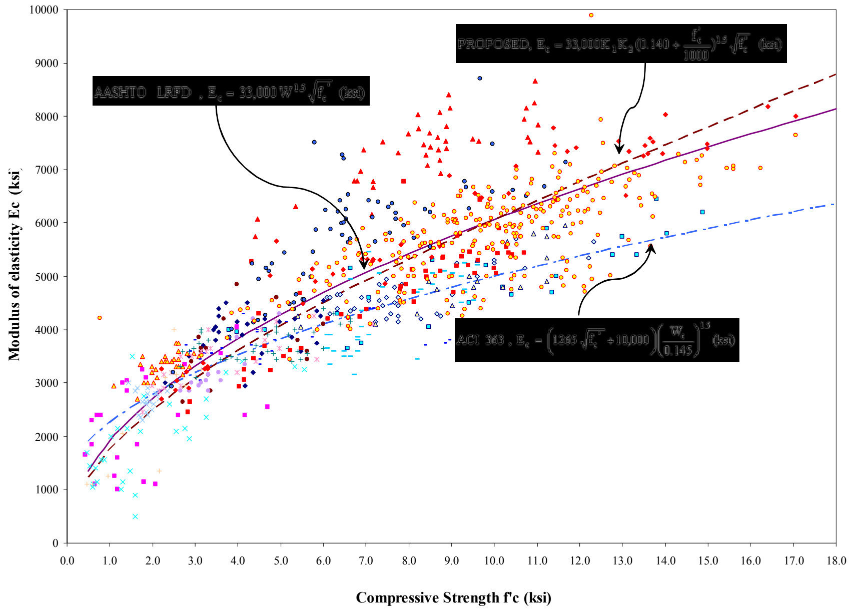

Fig. 1 NCHRP 18-07 Modulus of Elasticity Test Results

Another major point made in NCHRP 18-07[2] relevant to elastic effects is that when external loads, including initial prestress just before transfer to concrete and post-tensioning loads, are introduced to a *transformed* section, the elastic losses or gains are automatically accounted for. That is to say, when the change in concrete stress due to external loading is calculated using the transformed section rather than the net or gross sections, elastic effects need not be considered.

SHRINKAGE

Shrinkage is long-term loss caused by concrete's natural tendency to shrink over time. It is caused primarily by shrinkage of cement paste and depends on a number of variables including concrete strength, stiffness and proportion of aggregates, ambient conditions, and size and shape of the specimen. It continues throughout the service life of the bridge. Shrinkage strain of HSC can be different than normal concrete.

Twelve laboratory shrinkage specimens were tested in each of the four states. Three specimens were tested for each of the state's three high-strength girder mixes and three specimens were tested for each state's deck mix—a total of 16 mixes. One of the HSC mixes from each state was the same as the one used in that state's instrumented bridge girders. A total of 48 shrinkage specimens were tested in the lab.

In addition to the laboratory testing, three specimens were made from the same materials as Nebraska's girders and monitored in the field. They were subjected to the same curing and environmental conditions as the bridge girders.

The following is the NCHRP 18-07[2] proposed shrinkage formula:

$$\varepsilon_{sh} = 480x10^{-6} k_{td} k_s k_{hs} k_f \tag{3}$$

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} = \text{time development factor} \tag{4}$$

$$k_s = \frac{1064 - 94V/S}{735} = \text{size factor} \tag{5}$$

$$k_{hs} = 2 - 0.0143(H) = \text{humidity factor for shrinkage} \tag{6}$$

$$k_f = \frac{5}{1 + f'_{ci}} = \text{concrete strength factor} \tag{7}$$

Fig. 2 shows a summary of shrinkage test data from NCHRP 18-07[2] and compares current and proposed shrinkage prediction methods.

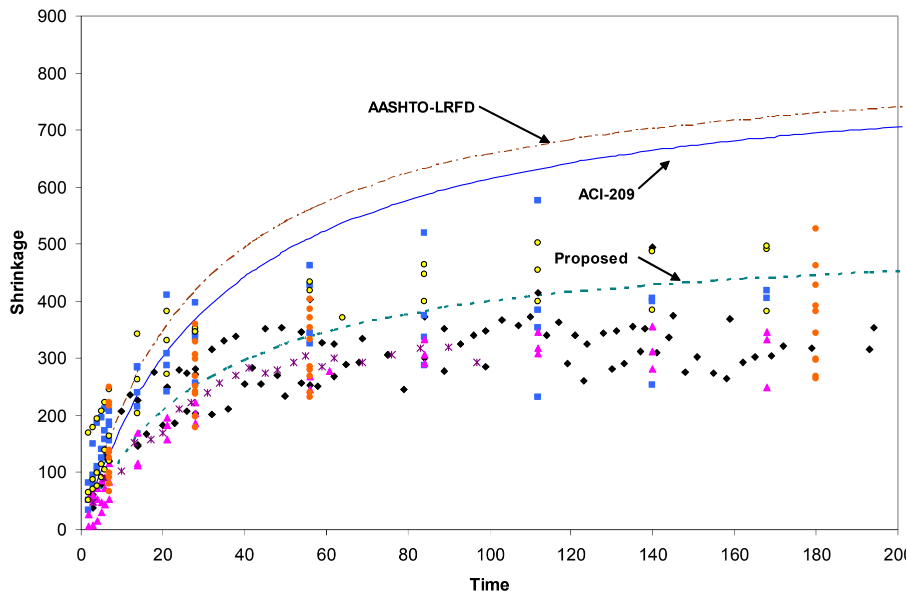

Fig. 2 NCHRP 18-07 Shrinkage Test Results

CREEP

Creep is a long-term loss caused by concrete's slow plastic deformation under prolonged stress. It depends on a number of variables including level of stress, duration of loading, strength and age of concrete, humidity, amount of steel reinforcement, cement content, w/cm ratio, aggregate proportions and properties, and type of curing and continues throughout the service life of the bridge. Creep in high-strength concrete is generally less than in normal-strength concrete.

NCHRP 18-07[2] included laboratory creep tests on all 12 high-strength mixes. Four 4″ x 4″ × 24″ specimens were tested for each mix. Three specimens were loaded at 1 day and one specimen was loaded at 56 days. A total of 48 creep tests were conducted.

The following is the NCHRP 18-07[2] proposed creep formula:

$$\Psi = 1.90 k_{td} k_{la} k_s k_{hs} k_f \tag{8}$$

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} = \text{time development factor} \tag{9}$$

$$k_{la} = t_i^{-0.118} = \text{loading age factor} \tag{10}$$

$$k_{hc} = 1.56 - 0.008H = \text{humidity factor for creep} \tag{11}$$

$$k_s = \frac{1064 - 94V/S}{735} = \text{size factor} \tag{12}$$

$$k_f = \frac{5}{1 + f'_{ci}} = \text{concrete strength factor} \tag{13}$$

Fig. 3 shows a summary of creep coefficient test data from NCHRP 18-07[2] and compares current and proposed creep prediction methods.

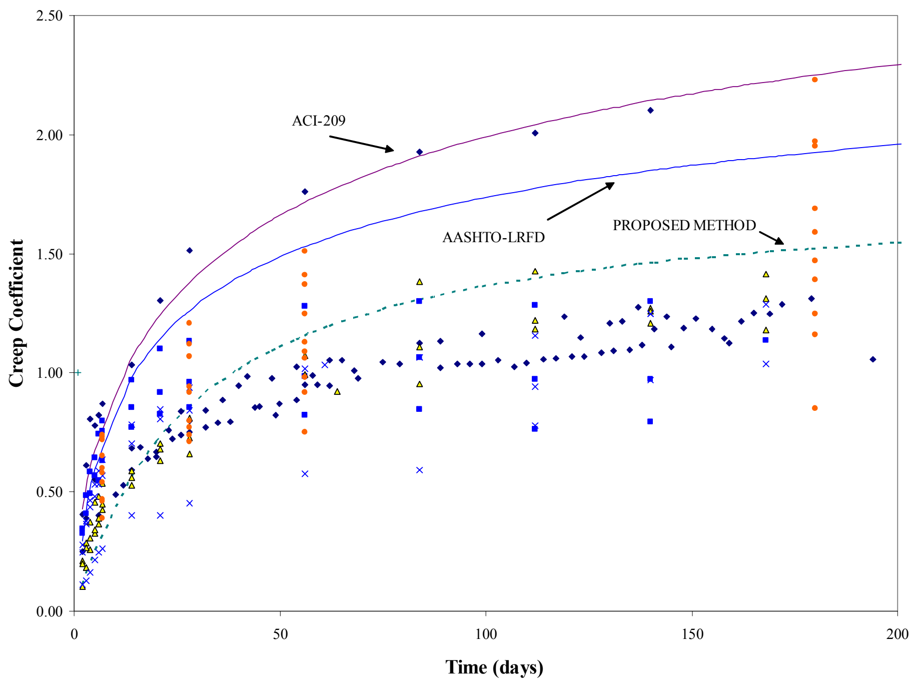

Fig. 3 NCHRP 18-07 Creep Coefficient Test Results (Specimens Loaded at 1 Day)

RELAXATION

The most commonly used type of prestressing steel on the market today is the low-relaxation strand. Their predecessor was the stress-relieved strand. Low-lax strands undergo an extra production step of controlled heating to about 660º F and then cooling while under tension. This reduces relaxation loss to about 25 percent of its predecessor. Due to widespread use of low-lax strands, relaxation effects are far less significant than in the past.

Typically, the relaxation loss of low relaxation strands ranges from 1.5 to 4.0 ksi. NCHRP 18-07[2] reasonably assumes this loss to be 2.4 ksi for the detailed method and 2.5 ksi for the approximate method.

TIME DEVELOPMENT CORRECTION FACTOR

The time development correction factor is used to estimate creep and shrinkage effects at any time. The following is the NCHRP 18-07[2] proposed time development correction factor:

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} \tag{14}$$

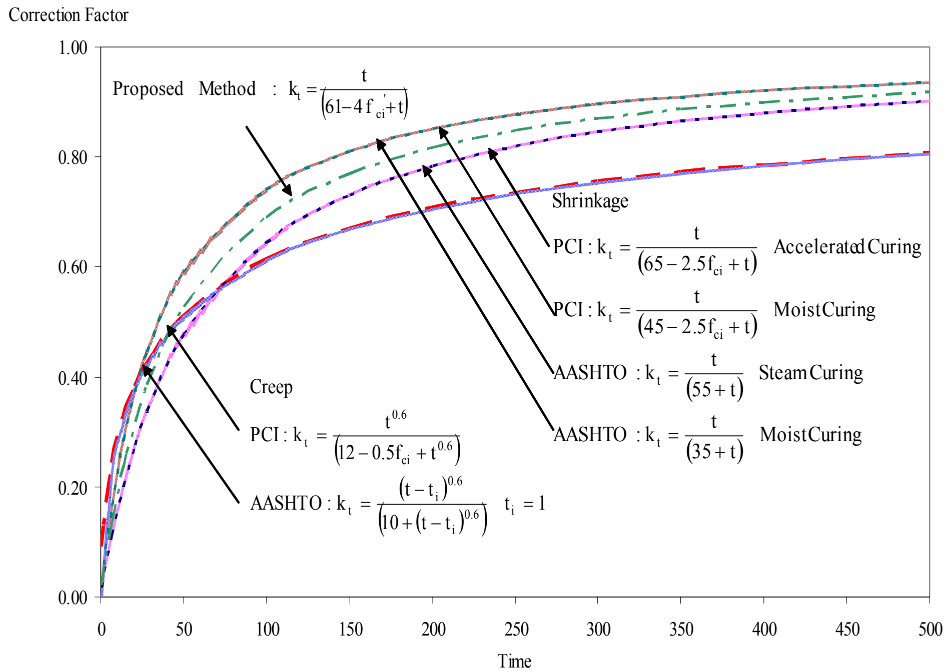

Fig. 4 Time Development Correction Factor by Various Methods

NUMERICAL EXAMPLE

The following numerical example is a single-span, single-stage post-tensioned bridge, designed in Nebraska.

SKYLINE DRIVE BRIDGE

The 198th – Skyline Drive Bridge in Omaha is a 63,000 mm (206 ft, 8.3 in) single-span bridge using NU2000PT Nebraska I-girders. The girder depth is 2,000 mm (6 ft, 6.6 in) and the web width is 175 mm (6.9 in). The bridge's cross section consists of 7 girders spaced at 2,550 mm (8 ft, 4.4 in). The bridge width is 17,686 mm (58 ft, 0.3 in). (Refer to Fig. 5.) The cast-in-place concrete slab is a composite 200 mm (8 in). There are three girder segments per girder line; the end segments are each 8,750 mm (28 ft, 8.5 in) and the field segment is 45,000 mm (149 ft, 3.3 in). These lengths allow for two splice joints. The release strength of the precast girders is 7.5 ksi. The 28-day compressive strength of the precast girders is 10 ksi and the slab is 4.3 ksi. The design live load is HL-93.

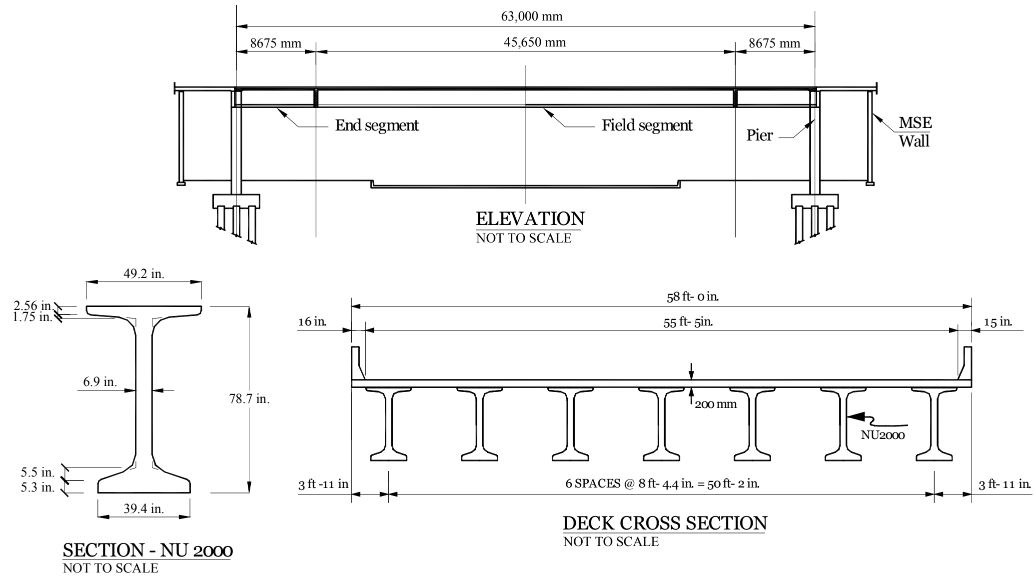

Fig. 5 General Layout of the 198th – Skyline Drive Bridge

Prestress losses were estimated using NCHRP 18-07[2]. Release occurred at 1 day, post-tensioning at 30 days, and deck placement at 60 days. The superimposed dead load was applied shortly after the deck placement. Relative humidity is 70 percent. The construction stages, as shown in Fig. 6, were as follows:

- **Stage 1**: Fabrication of precast girder segments.
- **Stage 2**: Erection of precast girder segments on temporary towers and abutments.
- **Stage 3**: Construction of splice joints.
- **Stage 4**: Post-tensioning and removal of the temporary towers.
- **Stage 5**: Placement of deck slab.
- **Stage 6**: Construction of barriers

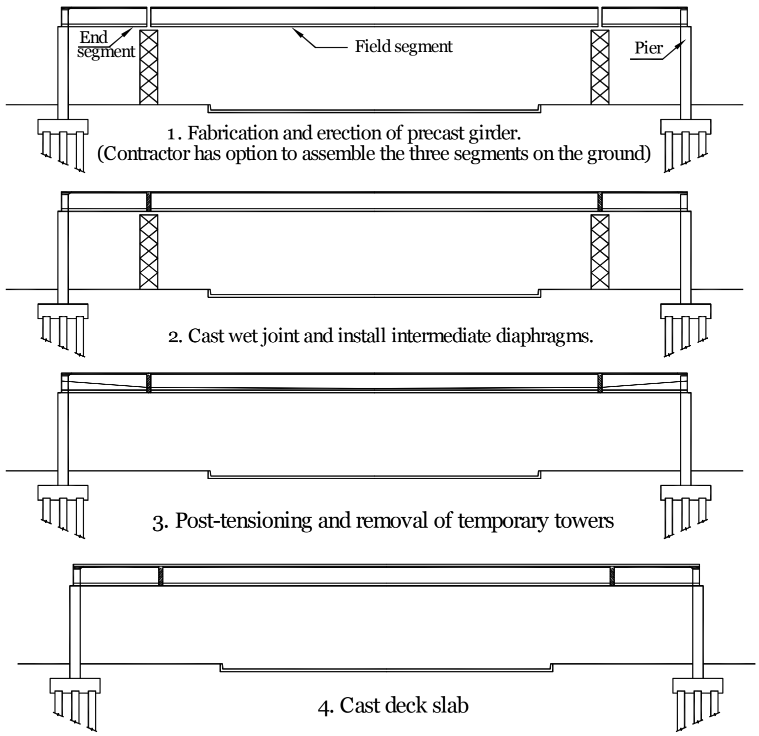

Fig. 6 Construction Sequence of the 198th – Skyline Drive Bridge

The critical section in flexure at final time due full loads plus effective prestress is at midspan. For pretensioning and post-tensioning details, refer to Figs. 7 and 8.

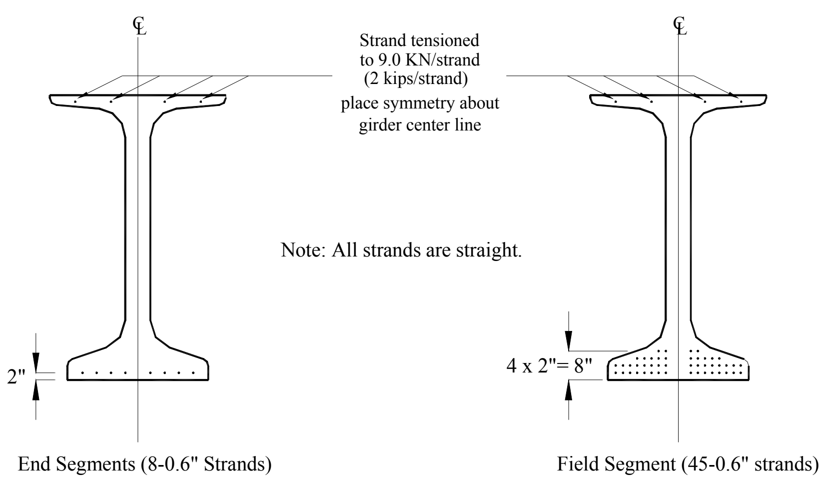

Fig. 7 Pretensioning Scheme of the 198th – Skyline Drive Bridge

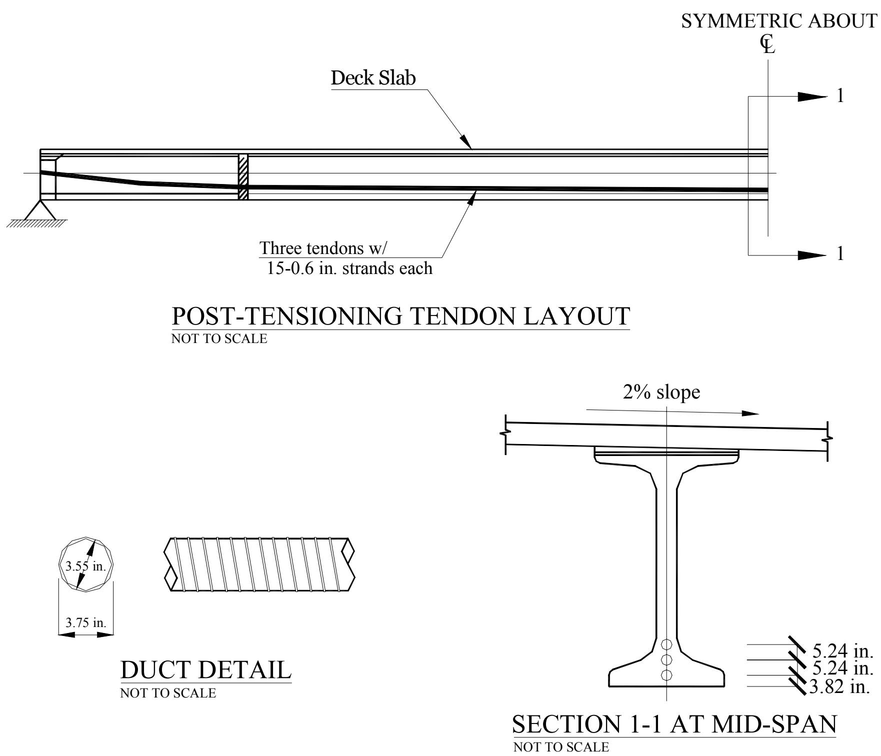

Fig. 8 Post-tensioning Details of the 198th – Skyline Drive Bridge

This bridge was designed using computerized time-dependent analysis, utilizing the CONSPLICE® software application[3]. Concrete material properties were specified as stated above and three different model codes (ACI[4], LRFD[5], and CEB-FIP-1999[6]) as well as NCHRP 18-07[2] equations were used.

PARAMETIC STUDY

The following table shows the parametric study, as well as the base case. The base case, as in cases 2, 5, and 8, have 10 ksi girder concrete strength, 70 percent relative humidity, post-tensioning at 30 days, deck placement at 60 days, and superimposed dead load at 67 days.

Table 1 Parametric Study

Parameter		
Girder Concrete Strength (ksi)	Case (1)	f'c=5 ksi, f'ci = 4 ksi
	Case (2)	f'c = 10 ksi, f'ci = 7.5 ksi
	Case (3)	f'c = 15 ksi, f'ci = 12 ksi
Relative Humidity (%)	Case (4)	40%
	Case (5)	70%
	Case (6)	80%
Time of D.L. Applications (days)	Case (7)	Deck placement 31 days, SDL at 38 days
	Case (8)	Deck placement 60 days, SDL at 67 days
	Case (9)	Deck placement 60 days, SDL at 180 days

COMPARISON OF RESULTS

Selected results were compared at the location of maximum moment (midspan). These results include pretensioned loss, post-tensioning loss, and concrete stress at the girder bottom fiber.

Figs. 9 and 10 show the time history of prestress and post-tensioning stresses through the final condition, estimated at about 10,000 days. Fig. 11 shows the concrete bottom fiber stress through the application of live load at final condition. These figures point to the lower long-term loss (creep and shrinkage) in high-strength concrete as evidenced by the NCHRP 18-07[2] predictions. Figs. 12 through 16 show the results at final condition when some conditions are varied.

Figs. 12 and 13 show the variations in the pretensioned strand prestress loss, post-tensioned tendons prestress loss, at final, at different girder concrete strengths. Fig. 14 shows the bottom fiber concrete stress at final with live load. Fig. 15 shows the variations in the bottom fiber concrete stress at final time, at various levels of relative humidity. Fig. 16 shows the variations in bottom fiber concrete stress at final varying the time of applying the superimposed dead load.

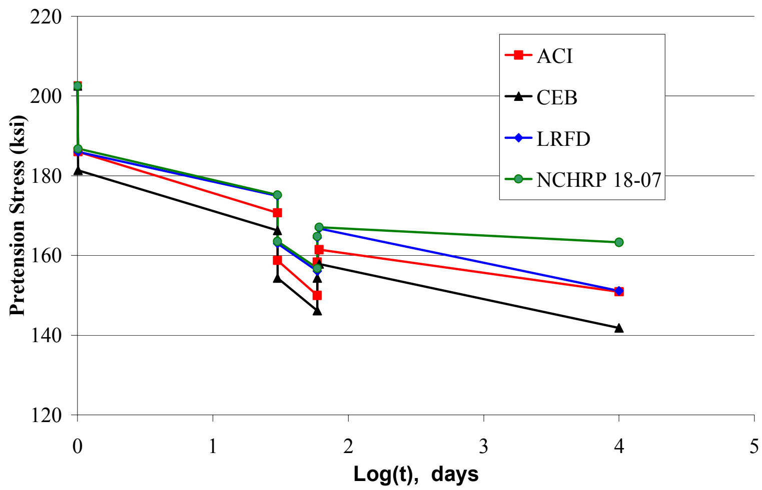

Fig. 9 Stress History in Prestressing Strands (Base Conditions; Cases 2, 5, and 8)

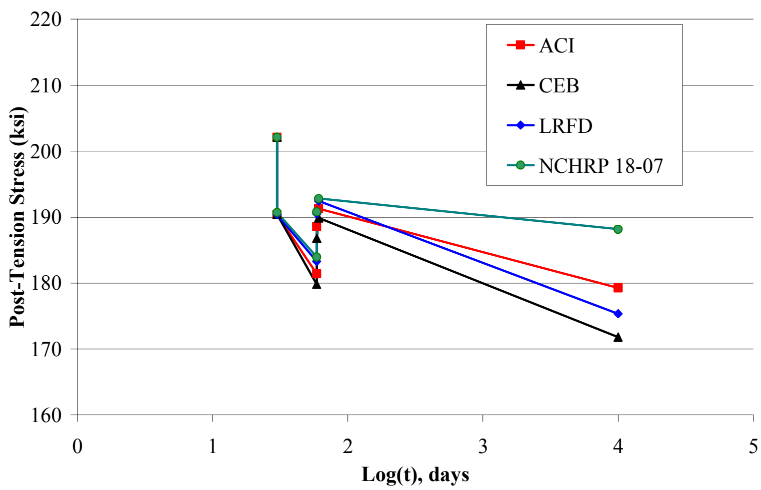

Fig. 10 Stress History in Post-Tensioning Tendons (Base Conditions; Cases 2, 5, and 8)

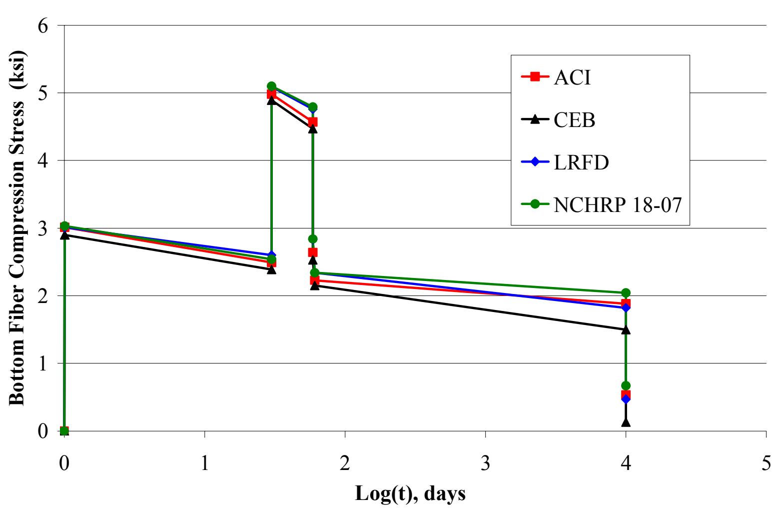

Fig. 11 Stress History in Bottom Fiber Concrete (Base Conditions; Cases 2, 5, and 8)

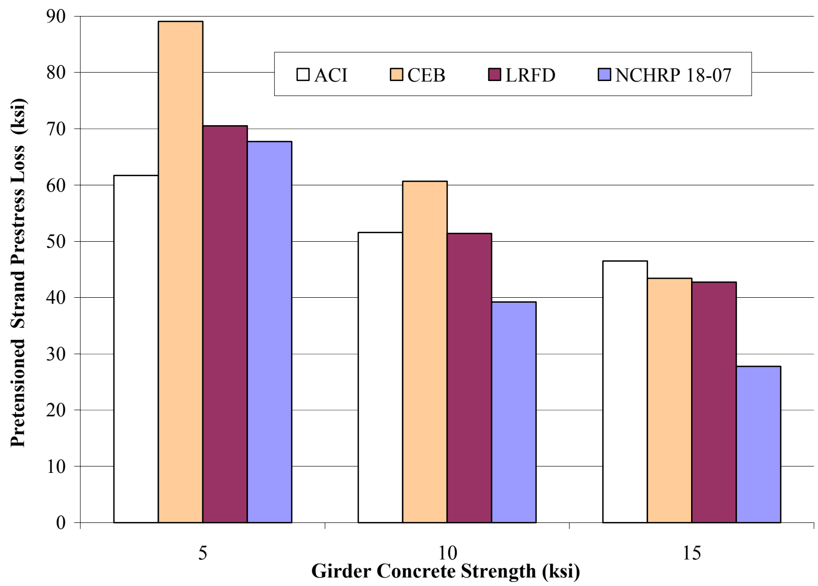

Fig. 12 Pretensioned Strand Prestress Loss for Varying Girder Concrete Strengths Using Various Methods

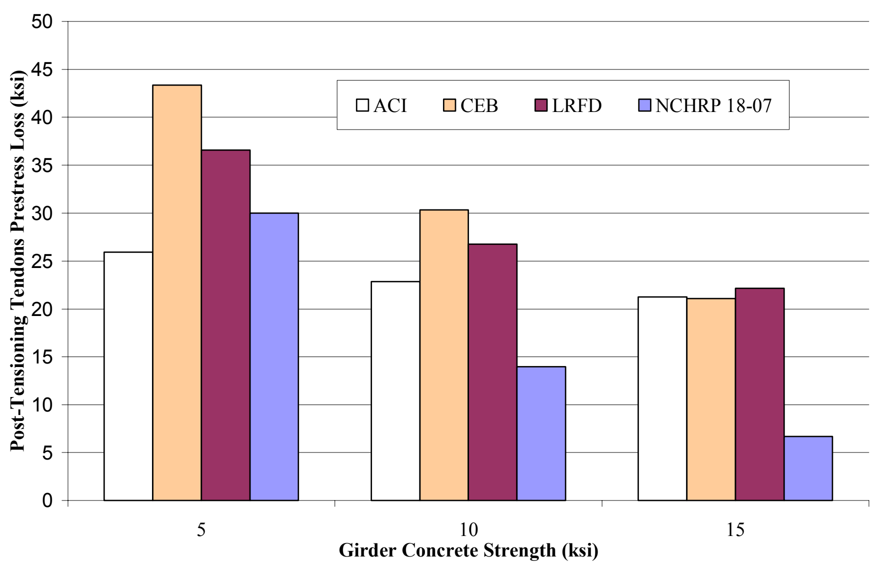

Fig. 13 Post-Tensioning Tendons Prestress Loss for Varying Girder Concrete Strengths Using Various Methods

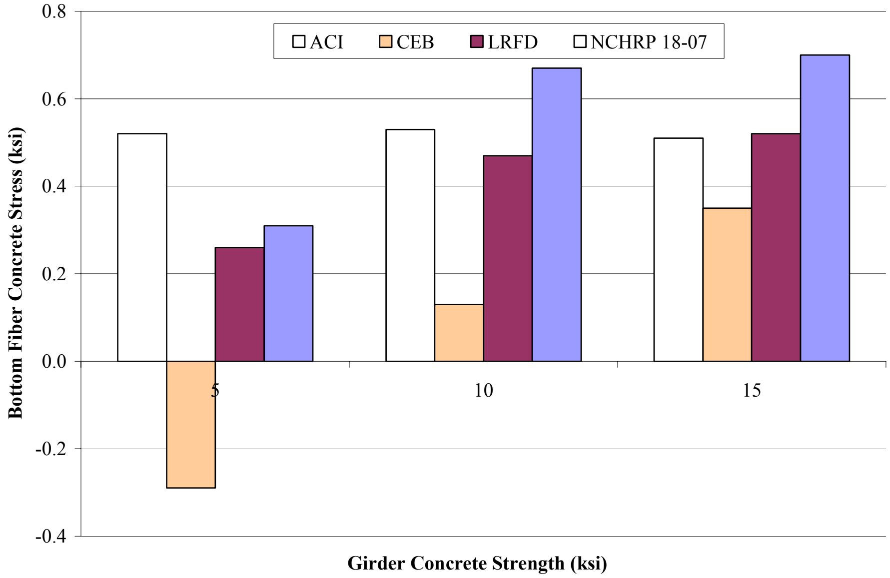

Fig. 14 Bottom Fiber Concrete Stress for Varying Girder Concrete Strengths Using Various Methods

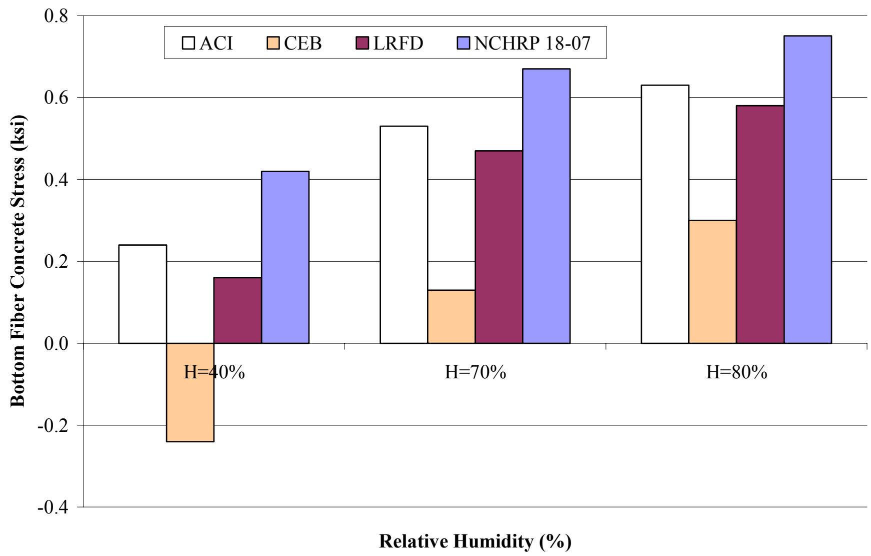

Fig. 15 Bottom Fiber Concrete Stress for Various Levels of Relative Humidity

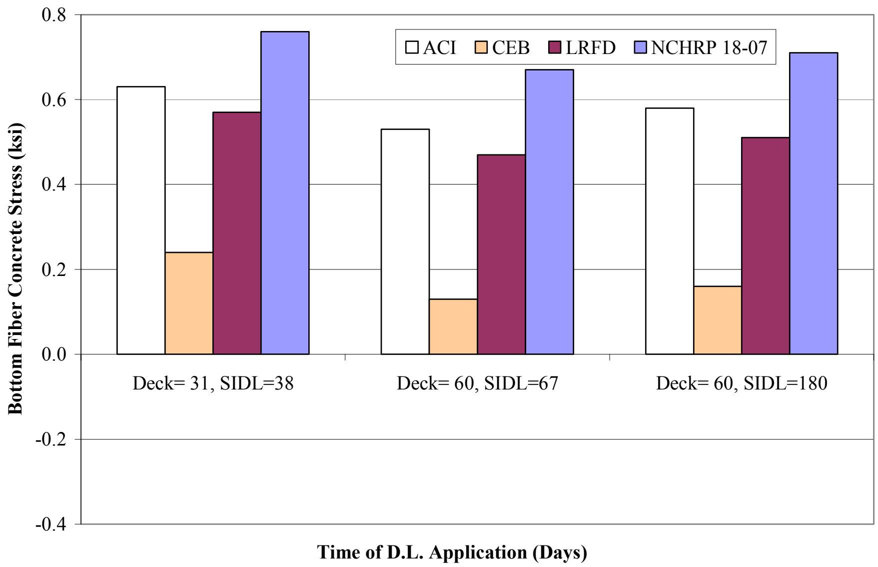

Fig. 16 Bottom Fiber Concrete Stress Varying the Time of Applying the SIDL

Examination of Figs. 12 through 16 reveals that high-strength concrete losses are overestimated in high-strength concrete when conventional equations are used. A study performed by Seguirant[7] (1998) on pretensioned girder bridges has demonstrated the same overestimation of pretensioned loss using LRFD equations. This leads to predictions that may be too conservative. The new formulas are also found to be accurate and in close agreement with the existing methods for normal strength concrete. The change in relative humidity can result in a variation in loss of prestress, with the trend being similar to traditional procedures, i.e., higher humidity causing lower loss of prestress. The timing of the application of composite loads is shown to have little effect on the final stresses, although what is not shown here is the fact that stresses at the time of loading can be affected.

CONCLUSIONS

The NCHRP 18-07[2] has resulted in new and improved equations for predicting the shrinkage and creep time-dependent effects in high-strength concrete. These equations are simpler and more straightforward to apply to computational methods and are proposed to replace the existing equations in AASHTO LRFD[5]. The new equations were examined here for a variety of conditions and were shown to be accurate for normal strength as well as high-strength concrete. It was also shown that their application to spliced girder bridges might be beneficial, as they result in more realistic predictions and more efficient designs.

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