Elasticity modulus, shrinkage, and creep of high-strength concrete as adopted by AASHTO

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In the past 20 years, the use of high-strength concrete (HSC) to improve the structural efficiency of pretensioned concrete girders has increased significantly. It is now standard practice to specify design concrete compressive strengths in excess of 8 ksi (55 MPa). In many regions, specifying 10 ksi to 12 ksi (69 MPa to 83 MPa) compressive-strength concrete results in little, if any, increase in girder cost compared with the standard 6 ksi (41 MPa) concrete used prior to the early 1990s.

HSC allows the use of greater levels of prestressing, thus increasing member span and spacing capabilities. Extrapolating the material property and prestress loss prediction methods developed for 4 ksi to 6 ksi (28 MPa to 41 MPa) concrete strengths to HSC has resulted in unrealistically high prestress loss estimates[1] and inaccurate camber and deflection predictions. A recent independent study by Stallings et al.[2] has confirmed that the pre-2005 American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications*[3] formulas for predicting long-term concrete material properties do not provide reliable estimates for HSC. There is a need for more accurate methods to estimate the material properties of HSC.

This paper covers the experimental and theoretical components of National Cooperative Highway Research Program (NCHRP) research project no. 18-07, which is discussed extensively in NCHRP report 496.[4] These components are

Editor's quick points

- Designers, owners, and precasters use high-strength concrete (HSC) because it can increase spans and spacing distances of members and reduce member section sizes.
- HSC's long-term material properties cannot be accurately extrapolated from those of normal-strength concrete.
- This paper includes results of research focused on the long-term material properties of HSC.

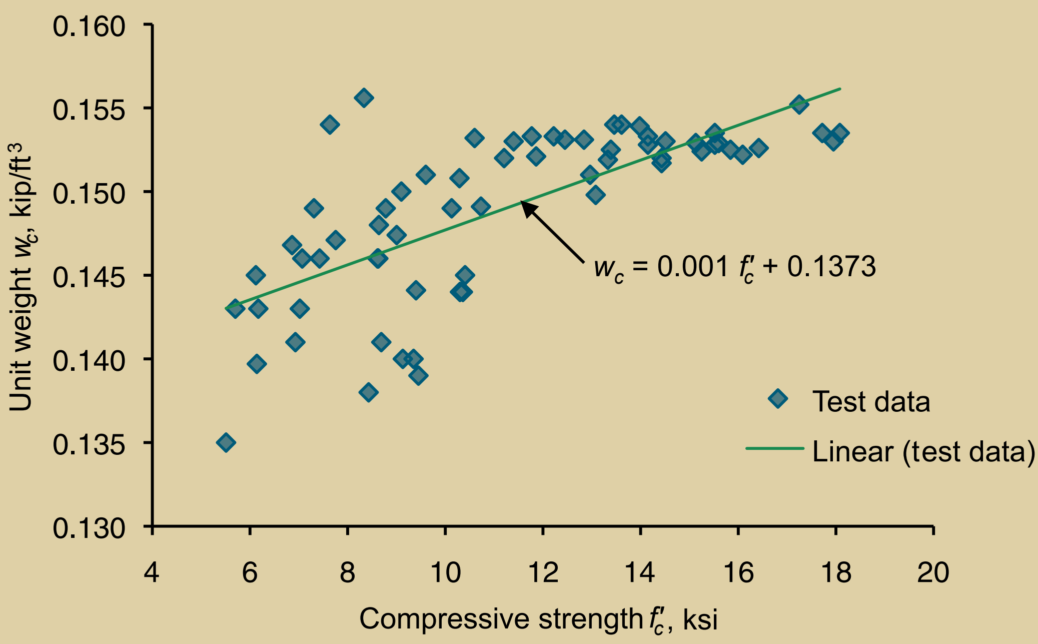

Figure 1. This graph shows the relationship between the density and compressive strength of concrete. Note: 1 ksi = 6.895 MPa; 1 kip/ft³ = 0.016 kg/m³.

related to modulus of elasticity, shrinkage, and creep of concrete. The experimental program was conducted at various bridge sites and at the University of Nebraska–Lincoln (UNL) for specimens produced from raw materials and mixture proportions provided by four participating states: Nebraska, New Hampshire, Texas, and Washington. These locations were selected for their geographic diversity to have a valid representation of U.S. materials and weather conditions. Previously reported measurements of material properties are also included.

The experimental program was used to extend the pre-2005 AASHTO LRFD specifications prediction formulas to concrete with compressive strengths up to 15 ksi (104 MPa). For each material property, a summary of the experimental values is presented followed by a comparison with the values obtained from the pre-2005 AASHTO LRFD specifications and the American Concrete Institute (ACI) 209 committee report.[5] The proposed formulas provide designers of prestressed concrete girders with more realistic estimates of long-term material properties, including effects of aggregate type and other significant factors. The use of the proposed formulas should give results comparable to those using the pre-2005 AASHTO LRFD specifications when concrete compressive strengths are close to 4 ksi (28 MPa). The use of these formulas with higher-strength concrete should result in more realistic camber predictions and lower prestress loss estimates.

The following sections present the background and recommendations for prediction of the modulus of elasticity, shrinkage, and creep of HSC. To help with clarity, the notation and units employed in the pre-2005 AASHTO LRFD specifications will be adopted as much as possible. Stresses will be expressed in units of ksi (MPa) rather than psi, as is generally used in *Building Code Requirements for Structural Concrete (ACI-318-99) and Commentary (ACI 318R-99)*.[6]

Influencing factors and pre-2005 AASHTO prediction

Modulus of elasticity

In North American practice, the modulus of elasticity of concrete has traditionally been considered to increase approximately with the square root of the compressive strength. Also, the modulus of elasticity has traditionally been assumed to vary with the density of concrete raised to the power of 1.5. This tradition was followed in this study. Equation (1) is the calculation for the modulus of elasticity of concrete E_c according to the pre-2005 AASHTO LRFD specifications Eq. (5.4.2.4.1):

$$E_c = 33{,}000 w_c^{1.5} \sqrt{f_c'} \tag{1}$$

where

f_c' = compressive strength of concrete

w_c = density of concrete

NCHRP report 496 has an equivalent equation in metric. The equation in section 8.5.1 of ACI 318 is identical to Eq. (1) except that the units are based on pounds rather than kips. The data used to develop this equation were based on concrete strengths ranging from 2 ksi to 7 ksi (14 MPa to 48 MPa). The data with relatively weak aggregate were omitted. The density of concrete ranged from 0.08 kip/ft^3 to 0.150 kip/ft^3 (1280 kg/m^3 to 2400 kg/m^3).

The ACI 363 committee report[7] indicates that Eq. (1) may overestimate the modulus of elasticity for compressive strengths over 6 ksi (41 MPa). That position was primarily based on the work of Carrasquillo.[8] The committee report recommends that the modulus of elasticity be estimated using Eq. (2).

$$E_c = \left(w_c / 0.145\right)^{1.5}\left(1000 + 1265\sqrt{f_c'}\right) \quad (2)$$

As will be seen from the correlation with test results, the authors have not detected any improvement in predicting E_c with Eq. (2). The research on which this paper is based was limited to normalweight concrete.

Many designers commonly use default values for the density of normalweight aggregate concrete when estimating E_c. These are generally assumed as 0.145 kip/ft^3 (2320 kg/m^3) for cast-in-place concrete and 0.150 kip/ft^3 (2400 kg/m^3) for precast concrete. However, concrete with relatively high strength has a low water-cement ratio and a relatively high density. Russell[9] has developed a best-fit relationship between density and strength (**Fig. 1**). The data further indicate that nearly all mixtures had a density less than 0.155 kip/ft^3 (2480 kg/m^3). This was later confirmed through a survey of the concrete producers in areas where dense aggregates are used. A simplified version of Russell's relationship can be used to represent the data in Fig. 1 with an upper limit of 0.155 kip/ft^3 (2480 kg/m^3) and a lower limit of 0.145 kip/ft^3 (2320 kg/m^3).

$$w_c = 0.140 + \frac{f_c'}{1000} \quad (3)$$

where

$0.145 \le w_c \le 0.155$

Equations (1) and (2) do not account for the effect of aggregate type. It has been observed[10,11] that stiff coarse aggregates can produce significantly higher modulus of elasticity for concretes of the same strength and density. As a result, the experimental work reported in this paper included identification of aggregate types and sources.

Shrinkage and creep

Shrinkage is influenced by factors such as volume-to-surface ratio, ambient relative humidity, concrete age, type of curing, and age of concrete under service. It is conveniently expressed as a dimensionless strain under uniform conditions of relative humidity and temperature. The pre-2005 AASHTO LRFD specifications provided formulas for estimating shrinkage.

For accelerated curing, shrinkage strain ε_{sh} is calculated from Eq. (4).

$$\varepsilon_{sh} = (560 \times 10^{-6})k_{td}k_s k_{hs} \quad (4)$$

where

k_{td} = time-development factor

k_s = size factor for the effect of the volume-to-surface ratio for shrinkage

k_{hs} = humidity factor for shrinkage

For moist curing, shrinkage strain ε_{sh} is calculated from Eq. (5).

$$\varepsilon_{sh} = (510 \times 10^{-6})k_{td}k_s k_{hs} \quad (5)$$

After one day to three days of accelerated curing, the time-development factor for shrinkage k_{td} is determined by Eq. (6).

$$k_{td} = \frac{t}{55 + t} \quad (6)$$

where

t = drying time after end of curing, days

After seven days of moist curing, the k_{td} and k_s are determined by Eq. (7) and (8), respectively.

$$k_{td} = \frac{t}{35 + t} \quad (7)$$

$$k_s = \left[\frac{\dfrac{t}{26e^{0.36V/S} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1064 - 94V/S}{923}\right] \quad (8)$$

where

V/S = volume-to-surface ratio of the exposed surfaces of the component

For average annual ambient mean relative humidity RH less than 80%, the humidity factor for shrinkage k_{hs} is calculated from Eq. (9).

$$k_{hs} = \frac{140 - RH}{70} \tag{9}$$

For RH greater than or equal to 80%, the humidity factor for shrinkage k_{hs} is calculated from Eq. (10).

$$k_{hs} = \frac{3(100 - RH)}{70} \tag{10}$$

The creep coefficient $\psi(t,t_i)$ is the ratio of creep strain occurring in the period t to the elastic strain at t_i caused by a constant stress applied to concrete of age t_i and sustained in the period t, where t is the age of concrete between time of loading for creep calculations, end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects and t_i is the age of concrete when load is initially applied. Creep strain will reach its ultimate value at the end of the service life of the structure. The creep coefficient is influenced by the same factors that influence shrinkage as well as the age of concrete at the time of loading. The coefficient is defined in such a way that the applied stress has to be a constant sustained stress within the levels that usually prevail for in-service conditions. It is not intended to be used for excessively high compressive stress. Structural analysis modeling allows use of the creep coefficient for cases where the stress in concrete varies with time, such as in the case of prestress losses and with deck or girder differential creep and shrinkage. Equations (11) through (16) are the pre-2005 AASHTO LRFD specifications creep-prediction formulas.

$$\psi(t,t_i) = 3.5 k_f k_c k_{hc} k_{la} k_{td} \tag{11}$$

where

$$k_f = \text{concrete strength factor} = \frac{1}{0.67 + \frac{f_c'}{9}} \tag{12}$$

k_c = size factor for creep

$$= \left[\frac{\dfrac{t}{26e^{0.36V/S} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1.80 - 1.77e^{-0.54V/S}}{2.587}\right] \tag{13}$$

k_{hc} = humidity factor for creep

$$= 1.58 - \frac{RH}{120} \tag{14}$$

Table 1. Laboratory and field materials testing program

Testing	Concrete age, days	Number of specimens				
		Deck	Girder			Field
		4 ksi	8 ksi	10 ksi	12 ksi	10 ksi
f_c' and E_c	1	3	3	3	3	3
f_c' and E_c	3	3	3	3	3	3
f_c' and E_c	7	3	3	3	3	3
f_c' and E_c	14	3	3	3	3	3
f_c' and E_c	28	3	3	3	3	3
f_c' and E_c	56	3	3	3	3	3
f_c' and E_c	90	3	3	3	3	n.d.
f_c' and E_c	128	3	3	3	3	n.d.
f_c' and E_c	256	3	3	3	3	n.d.
Shrinkage	7-day moist curing	3	n.d.	n.d.	n.d.	n.d.
	1-day accelerated curing	n.d.	3	3	3	3
Creep	1-day loading	n.d.	3	3	3	n.d.
	56-day loading	n.d.	3	3	3	n.d.

Note: E_c = modulus of elasticity of concrete; f_c' = specified compressive strength of concrete at 28 days unless another age is specified. n.d. = no data. 1 ksi = 6.895 MPa.

$$k_{la} = \text{loading age factor} = t_i^{-0.118} \quad (15)$$

$$k_{td} = \frac{(t - t_i)^{0.6}}{10 + (t - t_i)^{0.6}} \quad (16)$$

Experimental program

The experimental program consisted of materials testing in the laboratory and in the field, as well as testing of full-scale, high-strength prestressed concrete bridge girders in Nebraska, New Hampshire, Texas, and Washington. The following discussion is limited to a description of specimens used for evaluation of modulus of elasticity, creep, and shrinkage. Details of the girder testing will be covered in a subsequent paper.

The material testing program consisted of laboratory tests conducted at UNL and field tests conducted at production plants and construction sites. The concrete for each state included three HSC girder mixtures with design compressive strengths ranging from 8 ksi to 12 ksi (55 MPa to 83 MPa) and one normal-strength deck concrete with design compressive strength of 4 ksi (28 MPa). The precast concrete producer in each of the four states provided the mixture proportions and raw materials for production and testing of the specimens at UNL. In addition, each participating state highway agency provided the raw material and the mixture proportions for the deck concrete.

Specimens for testing compressive strength and modulus of elasticity were 4 in. × 8 in. (100 mm × 200 mm) cylinders. Creep and shrinkage specimens were 4 in. × 4 in. × 24 in. (100 mm × 100 mm × 600 mm) prisms. The concrete cylinders were made in accordance with ASTM

Table 2. Concrete mixture proportions

Mixture designation	Coarse aggregates			Fine aggregates		Water	Cement		Fly ash		Air, %
	Type	Size, in.	Weight, lb	Type	Weight, lb	Weight, lb	Type	Weight, lb	Class	Weight, lb	
NE04D	Limestone	1.50	883	Sand/gravel	2039	263	I	658	n.a.	n.a.	6
NE09G	Limestone	0.75	1530	Sand/gravel	1530	250	III	705	n.a.	n.a.	5-7
NE10G	Limestone	0.50	1860	Sand	990	240	I	750	C	200	5-7
NE12G	Limestone	0.375	1913	Sand/gravel	933	254	III	680	C	320	5-7
NE field	Limestone	0.75	1530	Sand/gravel	1530	250	III	705	n.a.	n.a.	5-7
NH04D	Gravel	1.00	1805	Sand	1205	250	II	658	F	132	2
NH10G	Gravel	0.75	1850	Sand	940	250	II	800	n.a.	n.a.	2
NH11G	Gravel	0.75	1850	Sand	925	250	II	800	n.a.	n.a.	2
NH12G	Gravel	0.75	1850	Sand	950	242	II	800	n.a.	n.a.	2
NH Field	Gravel	0.75	1850	Sand	940	250	II	800	n.a.	n.a.	2
TX04D	Gravel	0.75	1811	Sand/gravel	1192	244	I	611	C	152	2
TX08G	Limestone	0.75	2029	Sand	1237	206	III	611	n.a.	n.a.	2
TX09G	Limestone	0.75	2011	Sand	1340	192	III	564	n.a.	n.a.	2
TX10G	Limestone	0.75	1975	Sand	1237	197	III	705	n.a.	n.a.	2
TX field	Limestone	0.75	2011	Sand	1340	192	III	564	n.a.	n.a.	2
WA04D	Gravel	1.00	1810	Sand	1046	263	I	660	F	75	2
WA10G	Gravel	0.75	2010	Sand	1235	219	III	705	n.a.	n.a.	1.5
WA11G	Gravel	0.50	1877	Sand	1383	217	III	658	n.a.	n.a.	1.5
WA12G	Gravel	0.375	1959	Sand	1204	213	III	752	n.a.	n.a.	1.5
WA field	Gravel	0.75	2010	Sand	1235	219	III	705	n.a.	n.a.	1.5

Note: n.a. = not applicable. 1 in. = 25.4 mm; 1 lb = 0.453 kg.

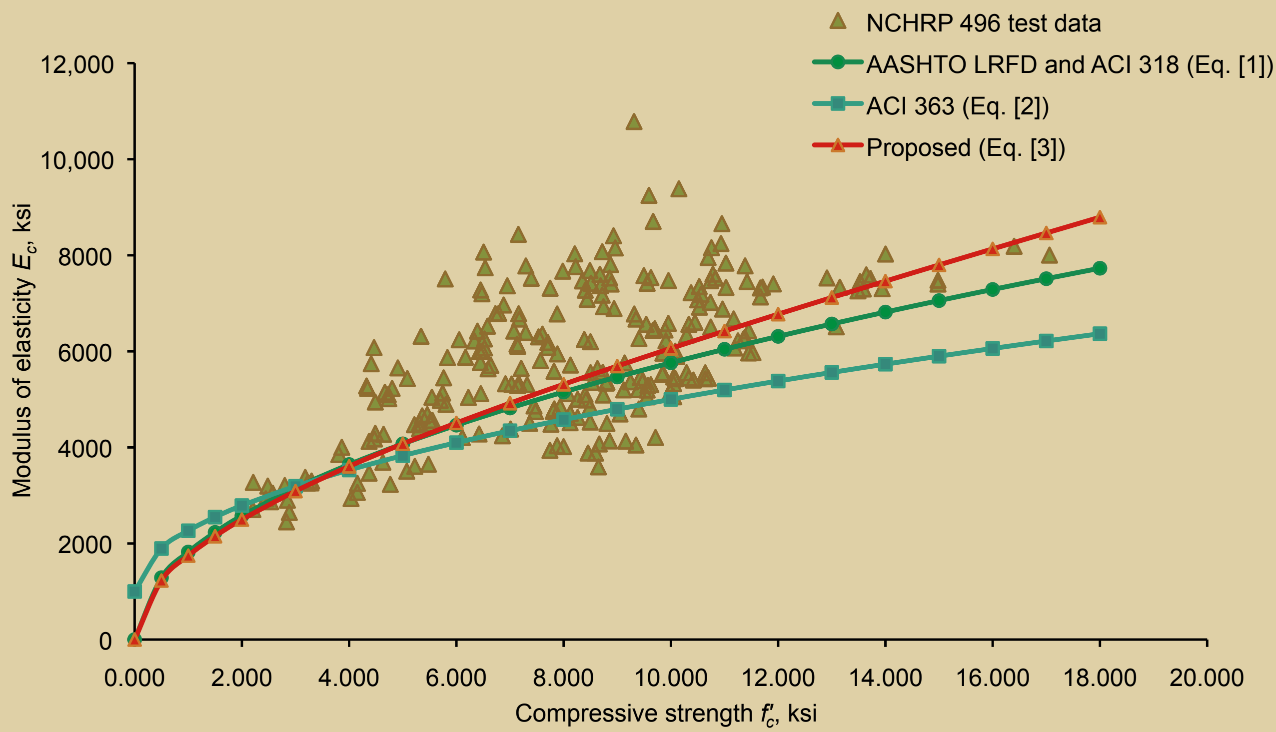

Figure 2. The modulus of elasticity was determined using results of experiments in this project. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program. 1 ksi = 6.895 MPa.

C192[12] and were cured in the laboratory curing room at an ambient temperature of 73 °F (23 °C) for 24 hr. **Table 1** summarizes the laboratory and field testing program. **Table 2** gives the mixture proportions of each concrete designation.

The testing for compressive strength, modulus of elasticity, shrinkage, and creep was performed according to ASTM C39,[13] ASTM C469,[14] modified ASTM C157,[15] and ASTM C512,[16] respectively.

All modulus-of-elasticity data were based on member-cured cylinders for field tests and moist-cured cylinders for laboratory tests. The on-site cured cylinders were subjected to the same long-term curing and storing conditions as those of the actual members, in accordance with ASTM C1231.[17]

The shrinkage specimens were cast at the same time and cured under the same conditions as the creep specimens. Readings were taken in parallel with the creep tests for each mixture to compare the time-dependent strain of loaded and unloaded specimens. The creep and shrinkage specimens in this project had a *V/S* of 1.0. The specimens were stored at an ambient *RH* of 35% to 40%.

Demountable mechanical (DEMEC) gauges were used to measure the surface strains in the longitudinal direction. Five DEMEC points were used on each of the two opposite faces of the specimens and were spaced at 4 in. (100 mm). This allowed for three 8 in. (200 mm) gauge lengths per surface, or six readings per specimen. Shrinkage readings were taken daily for the first week, weekly for the first month, and monthly for about a year.

Creep tests were performed in the laboratory on the twelve HSC mixtures. Similar to the shrinkage strain measurements, DEMEC gauges were used. A total of four specimens were made for each mixture. Three of these specimens were loaded at the age of one day, while the fourth was loaded at the age of fifty-six days. The specimens were loaded with an intensity of not more than 40% of the concrete compressive strength at the age of loading. The loading was initially applied using a hydraulic jack and measured with a load cell. Through nut tightening, the load was then transferred from the jack to the compressed spring. The level of sustained stress was kept constant through frequent measurements and adjustments.

The initial strain readings were taken immediately before and after loading. Creep measurements were then taken daily for the first week, weekly for the first month, and monthly for about a year. The creep coefficients were calculated from the measured total strains, elastic strains, and shrinkage strains.

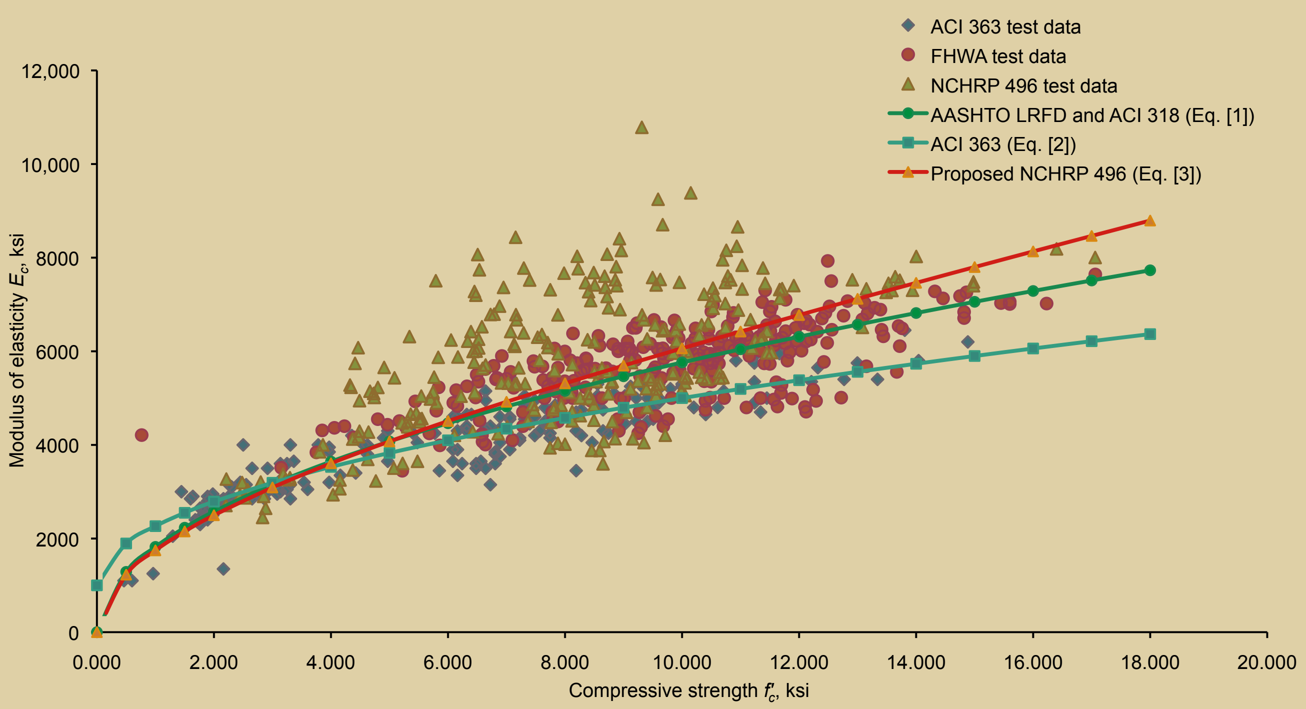

Figure 3. This graph shows the modulus of elasticity, including results of previous research. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; FHWA = Federal Highway Administration; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program. 1 ksi = 6.895 MPa.

Proposed prediction methods

The proposed prediction methods are presented as adopted by the AASHTO LRFD interim specifications for 2005[18] and 2006.[19] The same provisions appear in the fourth edition in 2007.[20] Slight modifications were made by the AASHTO subcommittee T10, Concrete Bridges, to the original proposal presented by the authors in NCHRP report 496. These modifications will be summarized in the following sections.

Proposed modulus-of-elasticity formula

There was considerable scatter in the modulus-of-elasticity data (**Fig. 2** and **3**). Introducing a variable density with concrete strength and an aggregate stiffness factor K_1 provides improvement to the results for HSC with unusually stiff or soft aggregates. Equation (17) is the proposed formula.

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'} \qquad (17)$$

where

K_1 = 1.0 unless determined by physical test and as approved by the authority of jurisdiction

w_c = $(0.140 + f_c'/1000)$
and 0.145 kip/ft^3 $\le w_c \le$ 0.155 kip/ft^3

where

f_c' = specified concrete compressive strength at service

The density of concrete w_c is assumed not to vary with time by taking f_c' constant. This is an improvement over the original proposal, where w_c varied with concrete strength as concrete aged. However, f_c' in Eq. (17) is variable with time. It is compressive strength of concrete at the same concrete age at which the modulus of elasticity is to be determined.

Proposed shrinkage- and creep-prediction formulas

Equations (18) and (19) are intended to represent the test data with a rectangular hyperbolic equation, similar to that in the ACI 209 committee report and the pre-2005 AASHTO LRFD specifications but with modifications to account for the effects of HSC.

$$\varepsilon_{sh} = (480 \times 10^{-6}) k_{td} k_{vs} k_f k_{hs} \qquad (18)$$

where

k_{vs} = a factor for the effect of volume-to-surface ratio

$$\psi(t, t_i) = 1.90 k_{td} k_{vs} k_f k_{hc} t_i^{-0.118} \qquad (19)$$

The ultimate creep coefficient was set at 1.90 for average conditions. This definition differs from that of the pre-2005 AASHTO LRFD specifications in which the ultimate creep coefficient was 3.5 for standard conditions. The difference between average and standard conditions will be discussed in the next section.

For example, average *RH* is 70% in the new provisions and standard *RH* is 40% in the pre-2005 AASHTO LRFD specifications. The correction factors in the new provisions are equal to unity for average conditions, while they were set equal to unity in the pre-2005 AASHTO LRFD specifications for standard conditions. A similar strategy was used to establish the ultimate shrinkage strain of 480×10^{-6} to represent the ultimate strain at average conditions. It is somewhat different from the values shown in Eq. (4) and (5) of 560×10^{-6} and 510×10^{-6} for accelerated and moist curing under standard conditions. Both the creep and shrinkage formulas yield results comparable to those of the pre-2005 AASHTO LRFD specifications for concrete strength at prestress transfer of 4.0 ksi (28 MPa), assumed in this paper to be equal to about 5.0 ksi (35 MPa) at 28 days, if other influencing factors are unchanged.

Proposed correction factors for shrinkage and creep under nonstandard conditions

Correction factors were used in the pre-2005 AASHTO LRFD specifications methods to modify the values of ultimate shrinkage and creep for any periods shorter than full service life and for nonstandard conditions. These standard conditions, in some methods, referred to laboratory specimen sizes and environmental conditions. For example, the ACI 209 committee-report method and the pre-2005 AASHTO LRFD specifications shrinkage-prediction methods consider an *RH* of 40% to be a standard condition, while most U.S. bridges are subjected to an average *RH* of about 70%. Also, the standard *V*/*S* was taken as 1.5 in. (38 mm) in the pre-2005 AASHTO LRFD specifications, while the average for most bridge members is about 3.5 in. (89 mm). The following correction factors have been reformatted to be equal to unity under average conditions.

Ambient relative humidity correction factor

Equations (20) and (21) are simplifications of the pre-2005 AASHTO LRFD specifications equations for shrinkage (Eq. [9] and [10]) and for creep (Eq. [14]). **Figure 4** shows a comparison of the various prediction methods normalized to unity at an *RH* of 70%. This figure shows two trends when normalized to a default value of 1.0 at

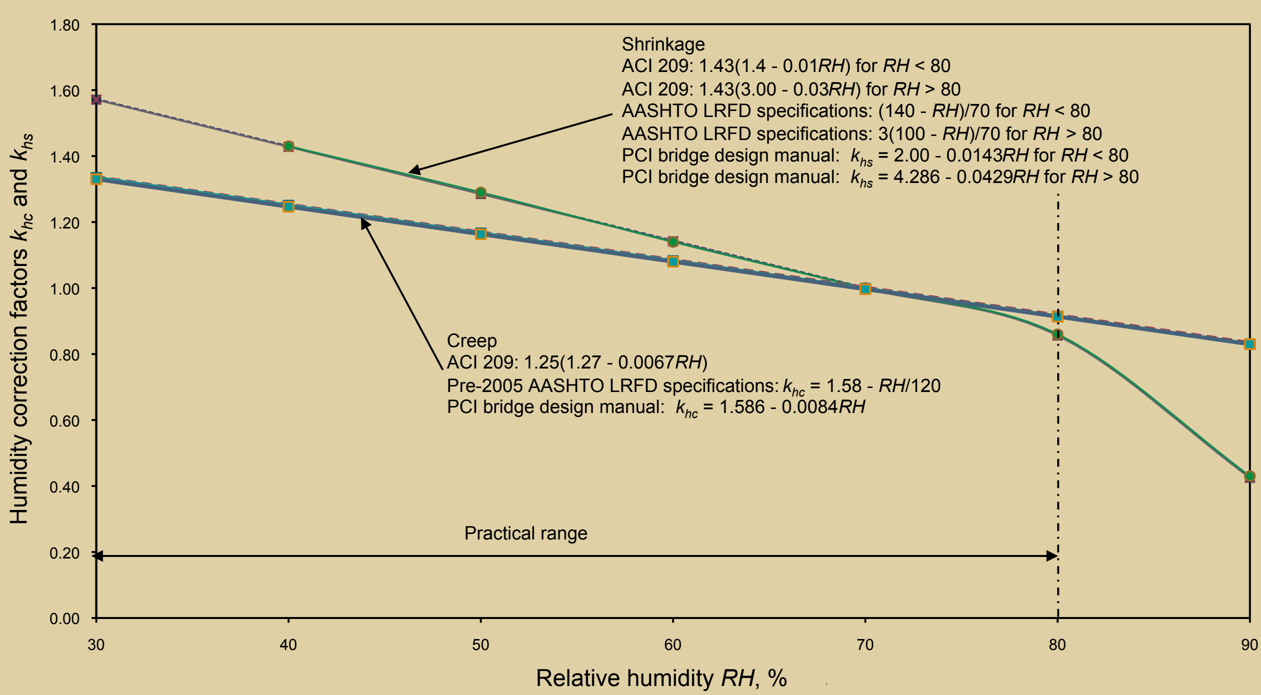

Figure 4. This graph determines the humidity correction factor according to various prediction methods. Note: Values predicted by various methods are normalized to unity at a humidity of 70%. AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; LRFD = load- and resistance-factor design.

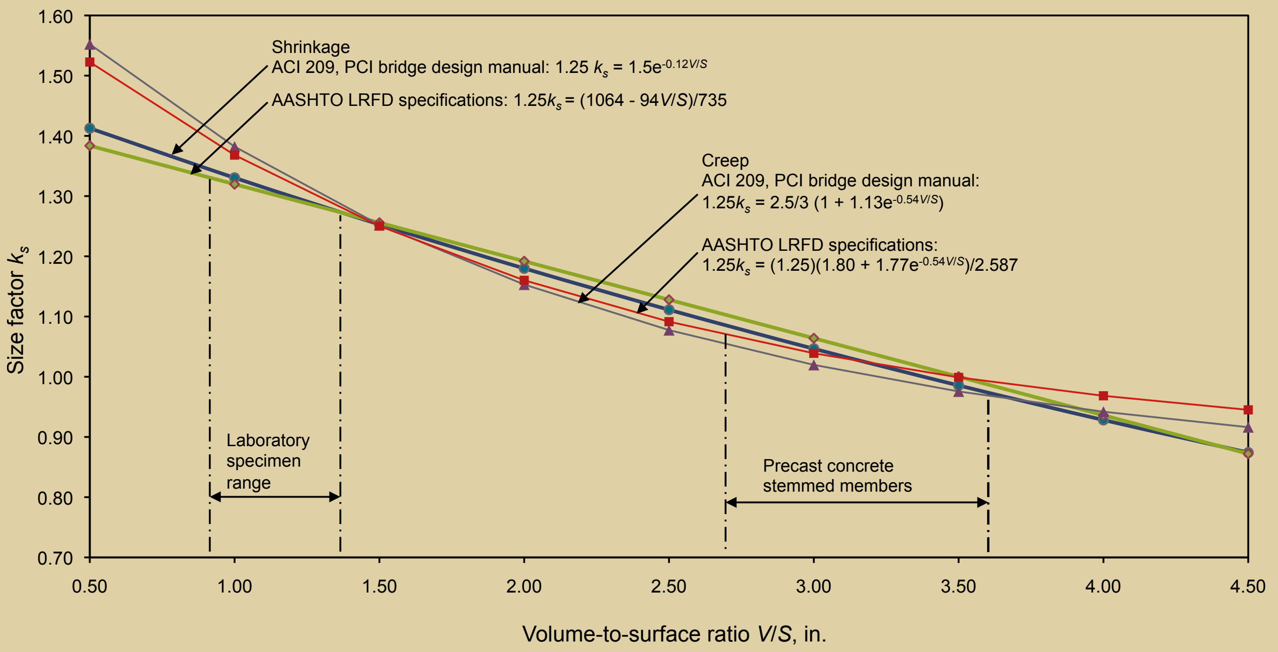

Figure 5. This graph determines the size correction factor according to various methods. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; LRFD = load- and resistance-factor design. 1 in. = 25.4 mm.

70% *RH*. Because the great majority of applications fall in the range of 30% to 80% ambient *RH*, the relatively low shrinkage coefficient for humidity higher than 80% is proposed to be conservatively ignored. This allows for reduction of the correction factor to just one formula for shrinkage (Eq. [20]) and another for creep (Eq. [21]).

$$k_{hs} = 2.00 - 0.014RH \tag{20}$$

$$k_{hc} = 1.56 - 0.008RH \tag{21}$$

Size correction factor

Relatively thick members do not dry as rapidly as thin members when subjected to ambient air with humidity less than 100%. This effect is captured by using the *V/S* factor. Member size affects short-term creep and shrinkage more than it does the ultimate values. The ultimate values are the ones of primary importance for stringer-type bridges. The size-factor formula is proposed to be simplified by using a time duration equal to infinity. **Figure 5** shows a comparison of the correction factors according to the pre-2005 AASHTO LRFD specifications, PCI's *Precast Prestressed Concrete Bridge Design Manual*,[21] and the ACI 209 committee-report formulas normalized for *V/S* equal to 3.5 in. (89 mm). This ratio corresponds to that for an I-girder with a web width of about 7 in. (180 mm).

The three formulas produce close results when used for the common range of *V/S*. Thus, it is proposed to use the simplest of the formulas (Eq. [8]) with the first bracketed term reduced to 1 due to the time *t* being taken equal to infinity:

$$k_{vs} = \frac{1064 - 94V/S}{735} = 1.45 - 0.13(V/S) \geq 0 \tag{22}$$

A lower limit of zero must be placed on k_{vs} to eliminate the possibility of irrationally using a negative shrinkage or creep for relatively thick members.

Loading-age correction factor

The pre-2005 AASHTO LRFD specifications and the ACI 209 committee-report prediction formulas were examined for computing the loading-age correction factor for both accelerated and moist curing conditions. **Figure 6** presents the correction factor for a range of loading ages normalized to a value of 1.0 for one day of accelerated curing or seven days of moist curing. This figure indicates that the variation of the correction factor with loading age follows a similar trend for both types of curing. Thus, the pre-2005 AASHTO LRFD specifications formula should continue to be used for both types of curing, with a shift in datum used to represent the difference in curing type. Accordingly, Eq. (15) is proposed for calculating the loading-age correction factor k_{la}.

$$k_{la} = t_i^{-0.118} \tag{15}$$

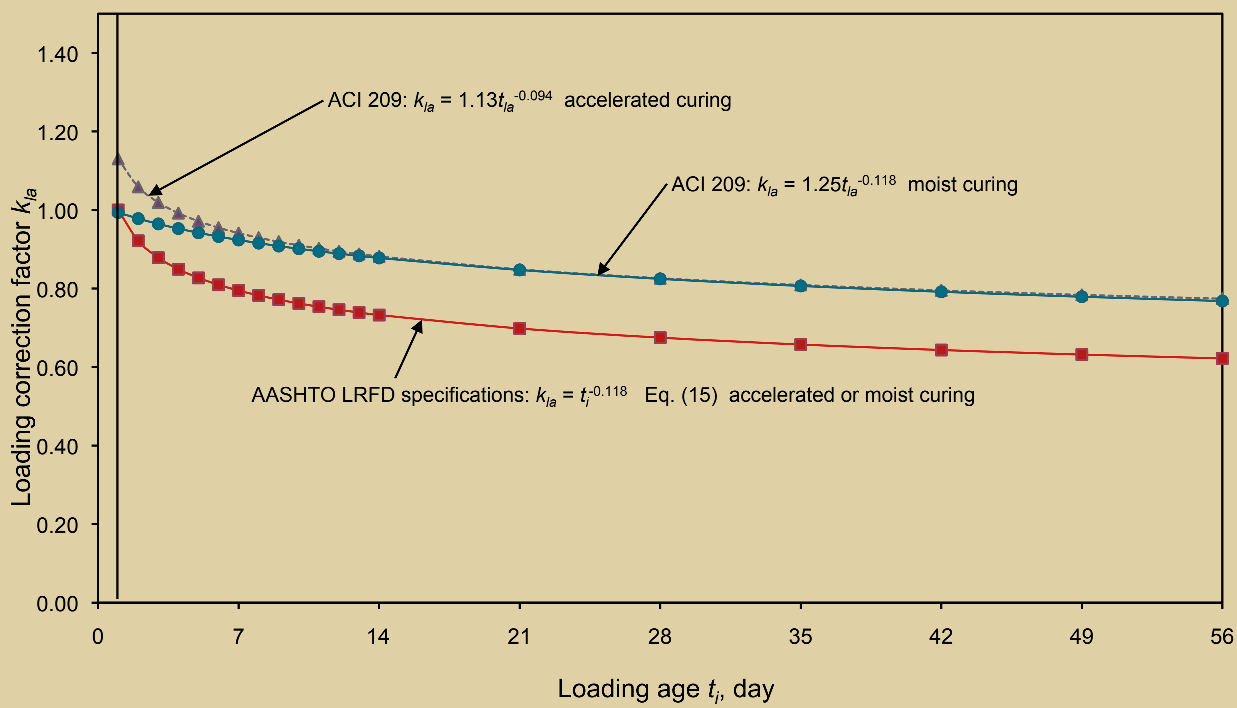

Figure 6. This graph determines the loading-age correction factor according to various methods. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; LRFD = load- and resistance-factor design; t_{la} = .

It is assumed that moist-cured concrete reaches the same level of maturity at seven days that accelerated-cured concrete reaches in one day. Thus, t_i is to be taken as equal to the actual concrete age for accelerated curing and the concrete age at the time of loading minus six days for moist-cured concrete loaded after a minimum of seven days of moist curing. Precast, prestressed concrete girders are generally assumed to have the first loading application at one day. That loading consists of the initial prestressing plus self-weight. Deck slabs that are made composite with the girders are assumed in the analysis to begin to interact with the girders after seven days of curing, creating differential shrinkage and creep. Additional load applications on the girder, namely deck weight and superimposed dead loads due to barriers and wearing surface, should be analyzed with t_i values corresponding to the actual age of the girder concrete.

Strength correction factor

The strength correction factor is one of the primary changes introduced in the new provisions. The ACI 209 and the pre-2005 AASHTO LRFD specifications shrinkage-prediction methods do not include a correction factor for concrete strength. The experimental results in this research clearly show the impact of HSC on reducing both creep and shrinkage. **Figure 7** shows a comparison of the correction factors according to the pre-2005 AASHTO LRFD specifications creep factor, Al-Omaishi,[22] and the proposed factor. Al-Omaishi demonstrated that creep and shrinkage should be more accurately related to concrete strength at the time of prestress release f'_{ci} than to the compressive strength at 28 days or 56 days.

The concrete strength factor obtained with the pre-2005 AASHTO LRFD specifications formula was normalized to a value of 1.0 for a final compressive strength in service f'_c of 5.0 ksi (35 MPa), with the assumed relationship $f'_c = (f'_{ci}/0.8)$. This assumption would allow usage of the same formulas in estimating creep and shrinkage of the deck slab, which has much less of an impact on the overall prestress loss and deformation of the bridge superstructure than does that of the girders. Therefore, the strength correction factor for both shrinkage and creep of concrete may be computed from Eq. (23).

$$k_f = \frac{5}{1 + f'_{ci}} \tag{23}$$

For nonprestressed members, f'_{ci} may be taken as $0.80 f'_c$.

Time-development correction factor

The time-development correction factor is used to estimate creep and shrinkage effects at times other than infinity. These effects are important in bridge design and construction if a relatively accurate camber prediction at the time

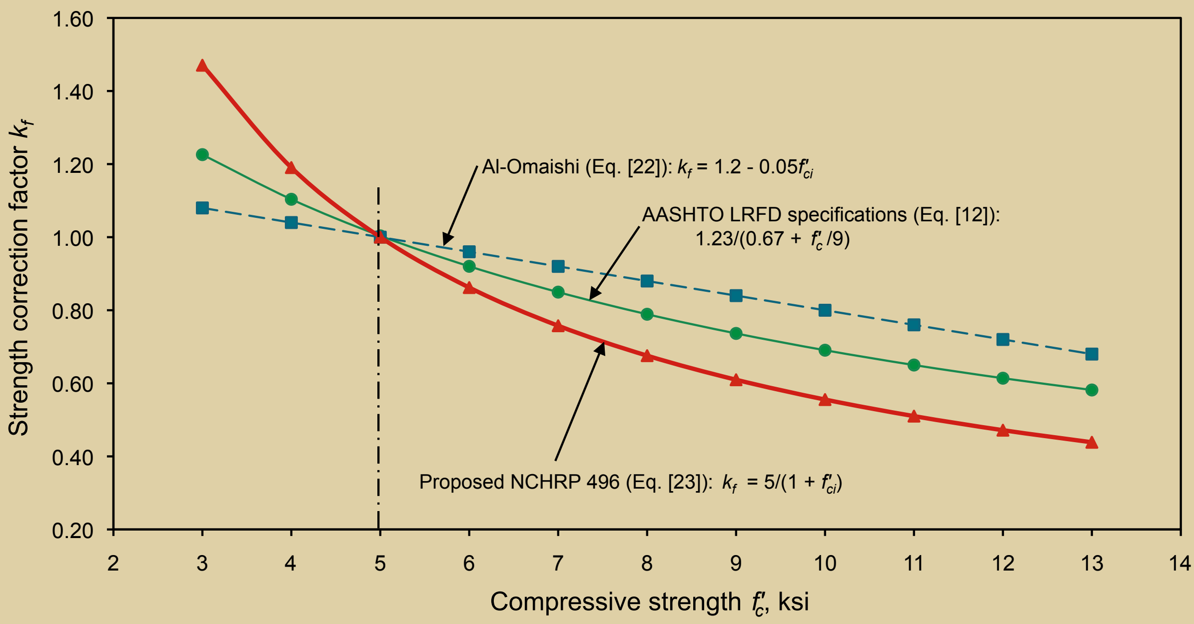

Figure 7. This graph shows the effect of different strength correction factors for different concrete compressive strengths. Note: Assume $f'_{ci} = 0.8f'_c$. AASHTO = American Association of State Highway and Transportation Officials; f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program. 1 ksi = 6.895 MPa.

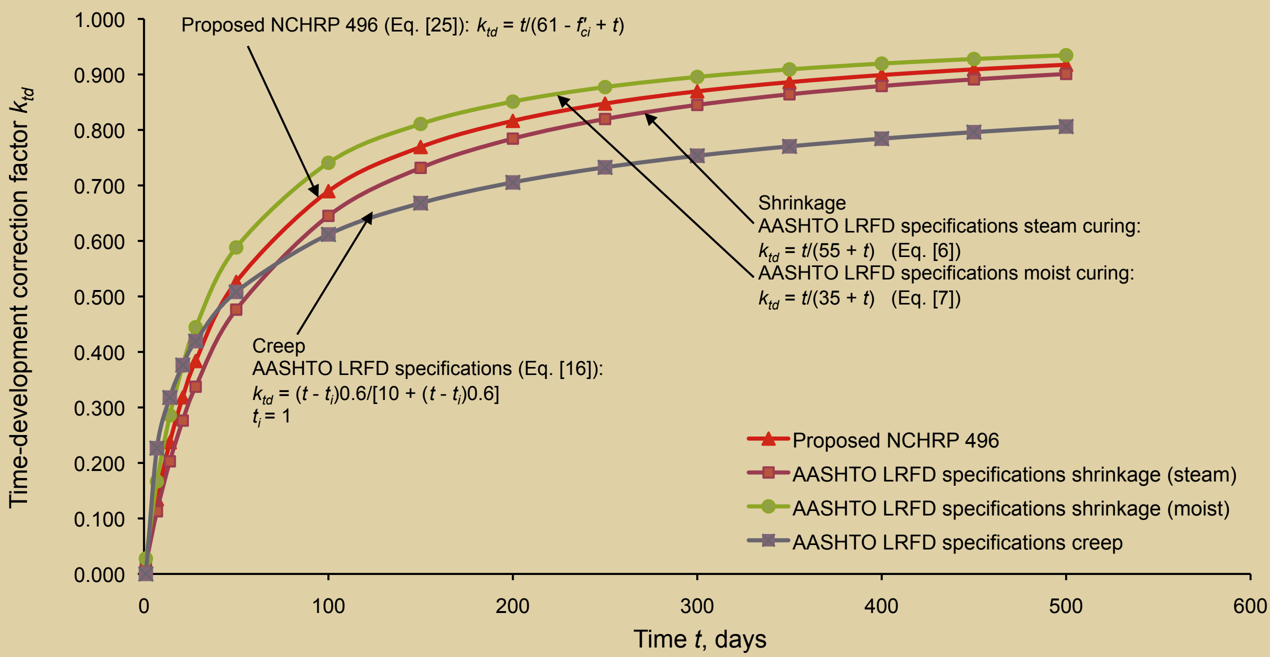

Figure 8. This graph shows the time-development correction factor by various methods. Note: AASHTO = American Association of State Highway and Transportation Officials; f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program; t = age of concrete between time of loading for creep calculations or end of curing for shrinkage calculations and time being considered for analysis of creep or shrinkage effects; t_i = age of concrete when load is initially applied.

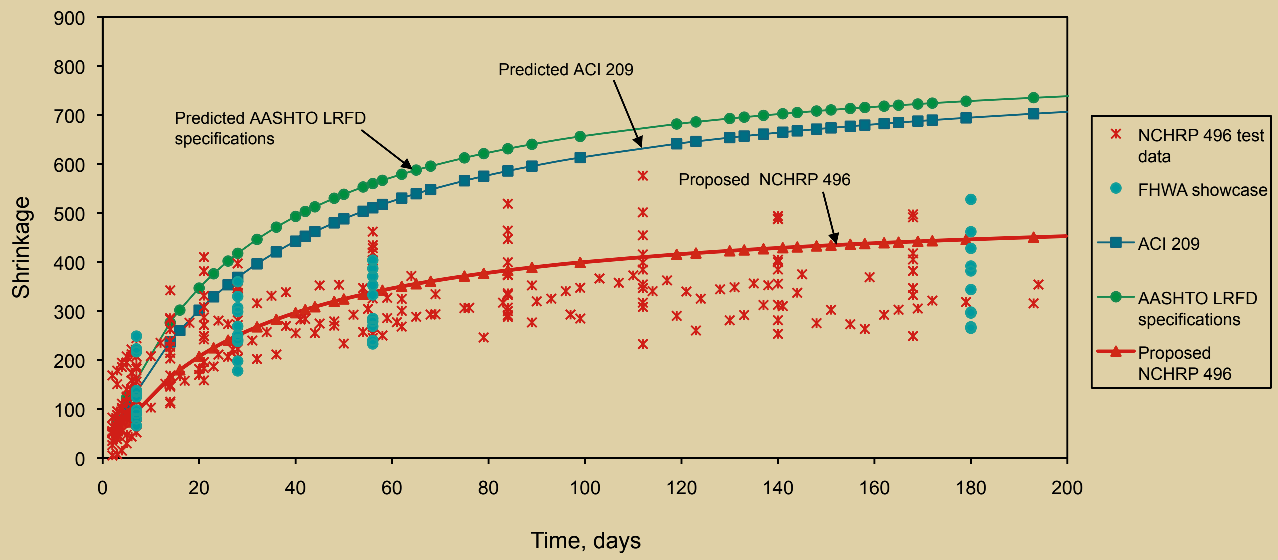

Figure 9. This graph shows the experimental results of shrinkage. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; FHWA = Federal Highway Administration; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program.

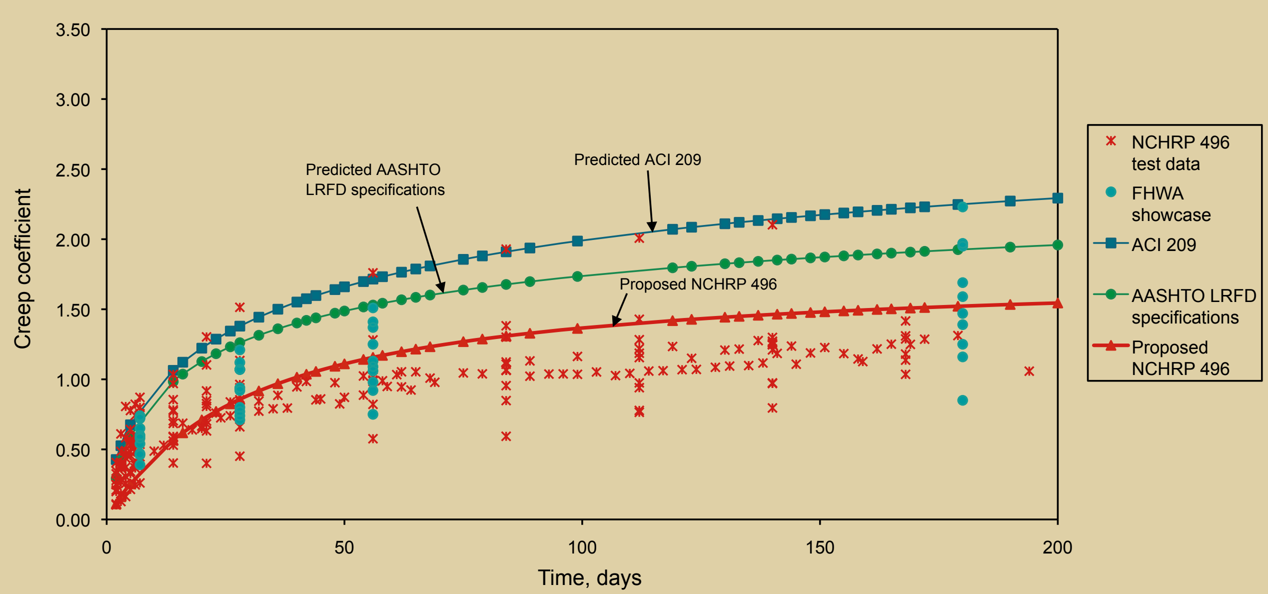

Figure 10. This graph shows the experimental results of creep. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; FHWA = Federal Highway Administration; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program.

of deck placement is to be made. The camber at that time is used to set girder seating elevations and to determine concrete haunch size and quantity over the girder and below the deck. This camber is becoming a significant design parameter with the increased use of HSC and corresponding high levels of prestress.

Figure 8 shows a comparison of the time-development correction factors calculated with various prediction methods. The pre-2005 AASHTO LRFD specifications and ACI 209 use the same time-development correction factor for predicting the shrinkage of concrete, but they use different formulas for creep. There are two formulas for shrinkage depending on type of curing: Eq. (6) and (7). Recent research presented in PCI's *Precast Prestressed*

Table 3. Ratios of predicted to measured modulus of elasticity of concrete E_c

Mixture	Coarse aggregate type	K_1	Ratio of predicted to measured E_c				
			AASHTO LRFD specifications/ACI 318		ACI 363	Proposed	
			w_c = 0.145 kip/ft^3	w_c = 0.150 kip/ft^3	w_c = 0.145 kip/ft^3	K_1 = 1.0	Variable K_1
Nebraska: NE04D, 09G, 10G, 12G, field	Crushed limestone	0.972	0.985	1.037	0.881	1.029	1.0
New Hampshire: NH04D, 10G, 11G, 12G, field	Gravel	0.910	1.066	1.122	0.958	1.099	1.0
Texas: TX04D, 08G, 09G, 10G, field	Crushed limestone	1.299	0.739	0.777	0.650	0.770	1.0
Washington: WA04D, 10G, 11G, 12G, field	Gravel	1.152	0.845	0.889	0.765	0.868	1.0
Average of participating states' data shown in Fig. 7			0.915	0.963	0.820	0.948	
Average of all data, including previous data shown in Fig. 8			0.987	1.037	0.875	1.020	

Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; f'_c = specified compressive strength of concrete at 28 days unless another age is specified; K_1 = aggregate-stiffness correction factor; LRFD = load- and resistance-factor design; w_c = density of concrete. 1 ft = 0.305 m; 1 lb = 0.453 kg.

Table 4. Ratios of predicted to measured shrinkage and creep coefficient

Mixture	Shrinkage strain			Creep coefficient		
	ACI 209	AASHTO-LRFD specifications	Proposed	ACI 209	AASHTO-LRFD specifications	Proposed
Nebraska: NE04D, NE09G, NE10G, NE12G, NE field	1.75	1.91	1.08	1.69	1.31	1.00
New Hampshire: NH04D, NH10G, NH11G, NH12G, NH field	1.13	1.27	0.80	1.50	1.37	0.84
Texas: TX04D, TX08G, TX09G, TX10G,TX field	2.26	2.60	1.57	2.06	1.89	1.08
Washington: WA04D, WA10G, WA11G, WA12G, WA field	1.05	1.18	0.74	1.89	1.88	0.99
Average of all data	1.55	1.74	1.05	1.79	1.61	0.98

Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; LRFD = load- and resistance-factor design.

Concrete Bridge Design Manual proposes possible modifications to account for concrete strength. As shown in **Fig. 9** and **10**, development of both shrinkage and creep is more accelerated at an early age in high-strength than in normal-strength concrete, in which development is more gradual over a longer period.

The proposed correction factor for time development of both shrinkage and creep for both conditions of curing is calculated from Eq. (24).

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} \quad (24)$$

where

t = age of concrete between time of loading for creep calculations or end of curing for shrinkage calculations and time being considered for analysis of creep or shrinkage effects

Equation (24) should not be used for concrete strength at release in excess of 12 ksi (82 MPa) and at service in excess of 12 ksi/0.8 (83 MPa/0.8) or 15 ksi (103 MPa).

Comparison of experimental results, prediction methods

Figure 2 shows the experimental results for modulus of elasticity from this research, while Fig. 3 combines the results with those from previous research, including those reported in the ACI 363 committee report and the Federal Highway Administration (FHWA) showcase projects.[23]

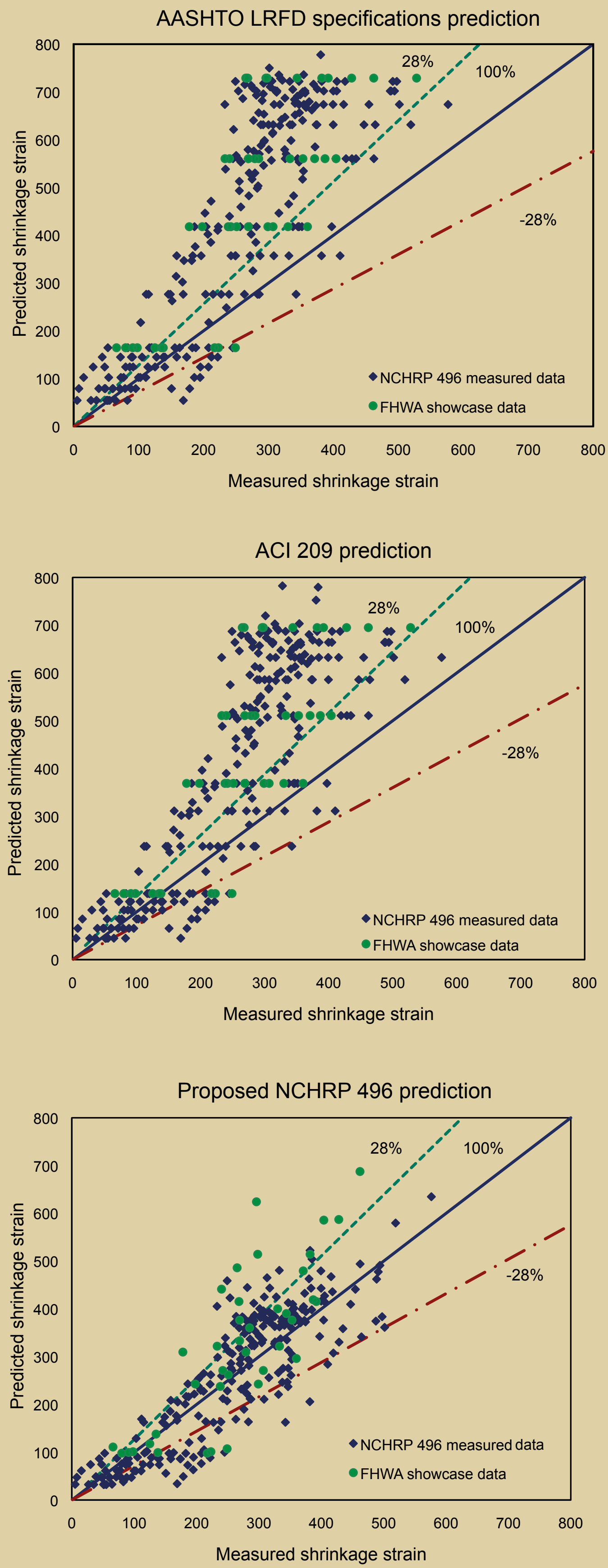

Figure 11. This graph compares measured with predicted values of shrinkage strain using AASHTO LRFD specifications, the ACI 209 committee report, and the proposed NCHRP 496 methods. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; FHWA = Federal Highway Administration; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program.

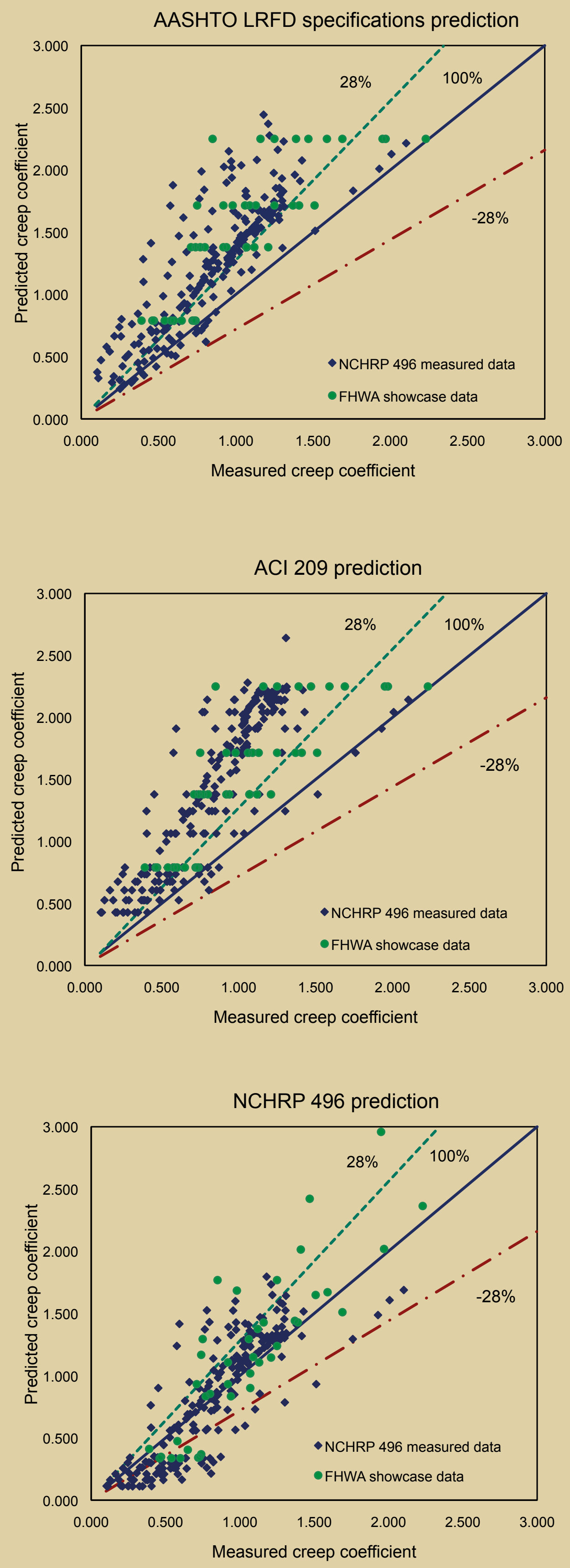

Figure 12. This graph compares measured with predicted values of creep coefficient using AASHTO LRFD specifications, the ACI 209 committee report, and proposed NCHRP 496 methods. Note: AASHTO = American Association of State Highway and Transportation Officials; ACI = American Concrete Institute; FHWA = Federal Highway Administration; LRFD = load- and resistance-factor design; NCHRP = National Cooperative Highway Research Program.

Figures 9 and 10 plot experimental shrinkage and creep results, respectively. More details are given in NCHRP report 496.

Table 3 gives a summary of ratios of predicted to measured modulus of elasticity by various methods. It shows that the average determined by the proposed formula using all test results is similar to the average determined by the pre-2005 AASHTO LRFD specifications and the ACI 318 formula. However, significant improvements are obtained if the aggregate stiffness factor K_1 is considered. The values of K_1 shown in Table 3 are only valid for aggregates similar to the corresponding aggregates used in the project. Otherwise, local testing must be performed to establish an appropriate value of K_1.

Figure 11 shows the measured versus predicted values of shrinkage using the pre-2005 AASHTO LRFD specifications, ACI 209, and proposed methods. Also shown in the figure are 28% upper- and lower-bound standard deviations that correspond to a 95% statistical confidence level. The figure demonstrates that the new method produces more-accurate predictions of the average, lower bound, and upper bound. **Figure 12** shows similar results for creep.

Table 4 compares the average ratios of predicted to measured values of shrinkage and creep by various methods. In general, the proposed method is in closer agreement with measured data than the other methods are.

Results

- The proposed formula for modulus of elasticity allows for variation in coarse aggregate type and stiffness as well as the effect of increasing density with increased concrete strength.
- The proposed shrinkage-prediction method produced results that averaged 105% of the measured values, compared with 174% when using the pre-2005 AASHTO LRFD specifications method and 155% when using the ACI 209 committee-report method.
- The proposed creep-prediction method produced results that averaged 98% of the measured values, compared with 161% and 179% for those estimated using the pre-2005 AASHTO LRFD specifications and the ACI 209 committee-report methods, respectively.

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References

1. Seguirant, S. J. 1998. New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders. *PCI Journal*, V. 43, No. 4 (July–August): pp. 92–119.

2. Stallings, J. M., R. W. Barnes, and S. Eskildsen. 2003. Camber and Prestress Losses in Alabama HPC Bridge Girders. *PCI Journal*, V. 48, No. 5 (September–October): pp. 90–104.

3. American Association of State Highway and Transportation Officials (AASHTO). 2004. *AASHTO LRFD Bridge Design Specifications*. 3rd ed. Washington, DC: AASHTO.

4. Tadros, M. K., N. Al-Omaishi, S. Seguirant, and J. Gallt. 2003. *Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders*. National Cooperative Highway Research Program (NCHRP) report no. 496. Washington, DC: Transportation Research Board, National Academy of Sciences.

5. American Concrete Institute (ACI) Committee 209. 1992. *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*. Detroit, MI: ACI.

6. ACI Committee 318. 1999. *Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (ACI 318R-99)*. Detroit, MI: ACI.

7. ACI Committee 363. 1992. *State of the Art Report on High-Strength Concrete*. Detroit, MI: ACI.

8. Carrasquillo, R. L., A. H. Nilson, and F. O. Slate. 1981. Properties of High Strength Concrete Subject to Short-Term Loads. *ACI Journal*, V. 78, No. 3 (May–June): pp. 171–178.

9. Russell, H. G. 2002. Personal communication.

10. Shideler, J. J. 1957. Lightweight-Aggregate Concrete for Structural Use. *Journal of the American Concrete Institute*, V. 29, No. 4 (October): pp. 299–328.

11. Myers, J. J., and R. L. Carrasquillo. 1999. The Production and Quality Control of High Performance Concrete in Texas Bridge Structures. Preliminary report no. 580-589-1, Center for Transportation Research, Austin, TX.

12. ASTM C192/C192M. 1992. *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*. West Conshohocken, PA: ASTM International.

13. ASTM C39. 2001. *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.* West Conshohocken, PA: ASTM International.

14. ASTM C469-94. 1994. *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*. West Conshohocken, PA: ASTM International.

15, ASTM C157. 2001. *Standard Test Method for Length Change of Hydraulic Cement Mortar and Concrete*. West Conshohocken, PA: ASTM International.

16. ASTM C 512. 2003. *Standard Test Method for Creep of Concrete in Compression.* West Conshohocken, PA: ASTM International.

17. ASTM C1231/C1231M. 2001. *Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders*. West Conshohocken, PA: ASTM International.

18. AASHTO. 2005. *AASHTO LRFD Bridge Design Specifications: 2005 Interim Revisions*. Washington, DC: AASHTO.

19. AASHTO. 2006. *AASHTO LRFD Bridge Design Specifications: 2006 Interim Revisions*. Washington, DC: AASHTO.

20. AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*. 4th ed. Washington, DC: AASHTO.

21. PCI Bridge Design Manual Steering Committee. 1997. *Precast Prestressed Concrete Bridge Design Manual*. 1st ed. Chicago, IL: PCI.

22. Al-Omaishi, N. 2001. Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders. PhD diss., Department of Civil Engineering, University of Nebraska, Lincoln, NE.

23. Federal Highway Administration (FHWA). 2002. *Compilation and Evaluation of Results from High Performance Concrete Bridge Projects*. Contract No. DTFH61-00-C-00009. Washington, DC: FHWA.

Notation

E_c = modulus of elasticity of concrete

f'_c = specified compressive strength of concrete at 28 days unless another age is specified

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing

k_c = size factor for creep

k_f = factor for the effect of concrete strength

k_{hc} = humidity factor for creep

k_{hs} = humidity factor for shrinkage

k_{la} = loading-age correction factor

k_s = size factor

k_{td} = time development factor

k_{vs} = factor for the effect of volume-to-surface ratio

K_1 = aggregate stiffness correction factor

= 1.0 unless determined by physical test and as approved by the authority of jurisdiction

RH = average annual ambient mean relative humidity

t = age of concrete between time of loading for creep calculations or end of curing for shrinkage calculations and time being considered for analysis of creep or shrinkage effects

t_i = age of concrete when load is initially applied

t_{la} = age of concrete when load is initially applied

V/S = volume-to-surface ratio of the member

w_c = density of concrete

ε_{sh} = shrinkage strain

$\psi(t,t_i)$ = girder creep coefficient at time t for loading at time t_i

Appendix: Numerical example

This example uses the data of example 9.4 of the *Precast Prestressed Concrete Bridge Design Manual*.[21] The bridge consists of 72-in.-deep (1.8 m) AASHTO-PCI bulb-tee girders spaced at 9 ft (2.7 m). The girders are designed to act compositely with the 8 in. (200 mm) cast-in-place concrete deck to resist the superimposed dead loads and live loads. The superimposed dead loads consist of the railing and a 2 in. (50 mm) future wearing surface. Both are assumed for this example to be introduced immediately after the deck has gained design strength. The cast-in-place haunch over the girder top flange is assumed to be 0.5 in. (13 mm) thick and 42 in. (1.1 m) wide.

The bridge is constructed in a region with relative humidity RH of 70%. Precast concrete strength at release f'_{ci} is 5.8 ksi and at service f'_c is 6.5 ksi. Cast-in-place concrete compressive strength at 28 days f'_c is 4.0 ksi. The aggregate stiffness factor K_1 is 1.0. Volume-to-surface ratio V/S is 3 for the precast concrete girder and 3.51 for the deck. The construction schedule allows for the following assumptions:

Concrete age at prestress transfer t_i is 1 day.

Age at deck placement t_d is 90 days.

Final conditions are assumed to occur at concrete age t_f of 20,000 days.

Material properties

Modulus of elasticity of concrete:

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

Girder at release:

$$E_c = (33{,}000)(1.0)\left(0.14+\frac{6.5}{1000}\right)^{1.5}\sqrt{5.8} = 4456 \text{ ksi}$$

Girder at final time:

$$E_c = (33{,}000)(1.0)\left(0.14+\frac{6.5}{1000}\right)^{1.5}\sqrt{6.5} = 4718 \text{ ksi}$$

Deck:

$$E_c = (33{,}000)(1.0)\left(0.14+\frac{4}{1000}\right)^{1.5}\sqrt{4} = 3607 \text{ ksi}$$

Creep

Girder

Creep coefficient at final time due to loading at transfer $\psi_b(t_f, t_i)$

$t = t_f - t_i = 20{,}000 - 1 = 19{,}999$ days

$k_{vs} = 1.45 - 0.13(V/S) = 1.45 - (0.13)(3) = 1.06 \geq 0$

$k_{hc} = 1.56 - 0.008RH = 1.56 - (0.008)(70) = 1.00$

$$k_f = \frac{5}{1+f'_{ci}} = \frac{5}{1+(5.8)} = 0.74$$

$$k_{td} = \left(\frac{t}{61-4f'_{ci}+t}\right) = \frac{19{,}999}{61-(4)(5.8)+19{,}999} = 1.00$$

$$\psi_b(t_f, t_i) = 1.9k_{vs}k_{hc}k_f k_{td} t_i^{-0.118}$$

$$= (1.9)(1.06)(1.00)(0.74)(1.00)(1)^{-0.118} = 1.48$$

Girder creep coefficient $\psi_b(t_d, t_i)$ at time of deck placement due to loading introduced at transfer:

$t_d = 90$ days, and $t = t_f - t_i = 90 - 1 = 89$ days

$$k_{td} = \left(\frac{t}{61-4f'_{ci}+t}\right) = \frac{89}{61-(4)(5.8)+89} = 0.70$$

$$\psi_b(t_d, t_i) = 1.9k_{vs}k_{hc}k_f k_{td} t_i^{-0.118} = (1.48)(0.7) = 1.04$$

Girder creep coefficient at final time due to loading at deck placement, $t_i = 90$ days

$$\psi_b(t_f, t_d) = 1.9k_{vs}k_{hc}k_f k_{td} t_i^{-0.118} = (1.48)(90)^{-0.118} = 0.87$$

Deck

$k_{vs} = 1.45 - 0.13(V/S) = 1.45 - (0.13)(3.51) = 0.99 \geq 0$

$k_{hc} = 1.56 - 0.008RH = 1.56 - (0.008)(70) = 1.00$

$$k_f = \frac{5}{1+f'_{ci}} = \frac{5}{1+(0.80)(4)} = 1.19$$

Deck creep at final time due to loads introduced shortly after deck placement:

$$\psi_b\left(t_f, t_d\right) = 1.9 k_{vs} k_{hc} k_f k_{td} t_i^{-0.118}$$

$$= (1.9)(0.99)(1.00)(1.19)(1.00)(1)^{-0.118} = 2.24$$

Shrinkage

Girder

Shrinkage strain between prestress transfer and final time:

k_{hs} = (2.00 – 0.014RH) = 2.00 – (0.014)(70) = 1.02

$\varepsilon_{bif} = k_{vs} k_{hs} k_f k_{td} 0.48 \times 10^{-3}$
= (1.06)(1.02)(0.74)(1.00)(0.00048)
= 0.000384 in./in.

Girder shrinkage strain between initial time and deck placement time, t = 90 – 1 = 89 days:

$$k_{td} = \left(\frac{t}{61 - 4f'_{ci} + t}\right) = \frac{89}{61 - (4)(5.8) + 89} = 0.70$$

$\varepsilon_{bid} = k_{vs} k_{hs} k_f k_{td} 0.48 \times 10^{-3}$ = (0.70)(0.000348)
= 0.000269 in./in.

Girder shrinkage strain between deck placement and final time:

$\varepsilon_{bdf} = \varepsilon_{bif} - \varepsilon_{bid}$ = 0.000384 – 0.000269 = 0.000115 in./in.

Deck

Shrinkage strain between end of deck curing and final time:

k_{hs} = (2.00 – 0.014RH) = 2.00 – (0.014)(70) = 1.02

$\varepsilon_{ddf} = k_{vs} k_{hs} k_f k_{td} 0.48 \times 10^{-3}$ = (0.99)(1.02)(1.19)(1.00)(0.00048)
= 0.000579 in./in.

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Synopsis

The use of high-strength concrete (HSC) for pretensioned concrete bridge girders has become commonplace among state highway agencies because of its economic and durability benefits. This paper summarizes part of the research work performed under the National Cooperative Highway Research Program (NCHRP) project 18-07, Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders, which is fully documented in NCHRP report no. 496. The researchers were assigned the task of extending the American Association of State and Highway Transportation Officials' (AASHTO's) *AASHTO LRFD Bridge Design Specifications* provisions for estimating prestress losses to cover concrete strengths up to 15 ksi (104 MPa).

This paper summarizes the portion of that work on concrete properties that have an impact on design for long-term effects: modulus of elasticity, shrinkage, and creep. These research findings were adopted into the 2005 and 2006 interim provisions of the AASHTO LRFD specifications. The experimental component of the research includes testing of specimens produced from raw materials and mixture proportions provided by four participating states (Nebraska, New Hampshire, Texas, and Washington) to encompass the regional diversity of materials throughout the country. The theoretical component of the research addresses the background of prior prediction formulas and the development of the new formulas that have now been adopted.

Keywords

Creep, high-strength concrete, HSC, material properties, modulus of elasticity, prestress loss, relaxation, shrinkage.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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