Investigation of measured prestress losses compared with design prestress losses in AASHTO Types II, III, IV, and VI bridge girders

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- This study investigates how measured prestress losses can deviate from design values and the reasons for this discrepancy.
- Prestress losses, compressive strength, modulus of elasticity, shrinkage, and creep were measured for several American Association of State Highway and Transportation Officials Types II, III, IV, and VI girders, including internal strain monitoring for nine full-scale girders.
- Measured concrete compressive strength was significantly higher than design compressive strength, which contributes to the difference between measured and design prestress losses.
- Recommendations to improve the estimation of concrete properties and prestress losses are provided.

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In the design of precast, prestressed concrete bridge girders, predicting prestress losses is essential for determining camber, critical stresses in the concrete, and the structural capacity of the girder. Accurately predicting prestress losses requires reasonable estimates of several parameters, which can be a challenging task at the design stage. For many reasons, the predicted prestress losses tend to deviate from measured values.[1,2] These reasons include differences between the design and the actual concrete properties, variations among local material properties, variations in the production circumstances, and the accuracy of the prediction methods. Several studies have confirmed that the properties of local materials affect the accuracy of estimates of elastic modulus, creep, and shrinkage of concrete.[3–5] Curing methods, concrete maturity at transfer, storage period, and storage conditions are all different among precasting plants. This influences prestress losses over time.

Overestimating prestress losses leads to higher-than-expected prestressing forces, larger-than-expected cambers, and an excessive number of strands. Underestimation of the prestress losses, on the other hand, may lead to insufficient prestressing force and cracking in the bottom surface of the girder under full service load. This study provides insight into how the measured prestress losses can deviate from the design values and investigates the reasons for this discrepancy. Field measurements included testing of concrete materials and internal strain monitoring for nine full-scale girders of different sizes and lengths. Prestress losses were monitored during fabrication, storage, and erection, and after deck placement. The study evaluated the accuracy of the eighth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[6] published in 2017 for prediction of prestress losses for AASHTO Types II, III, IV, and VI girders, which are widely used in highway bridges.

Background and motivation

The majority of prestress losses come from elastic shortening, creep, and shrinkage.[7,8] The accuracy of elastic shortening estimates depends mainly on the accuracy of the predicted modulus of elasticity of concrete at transfer. Predicting the creep and shrinkage losses is a more challenging task because they are affected by relative humidity, ambient temperature, aggregate content, aggregate properties, and water-to-binder ratio.[3,7,9] Moreover, accurate estimation of the prestress losses requires accounting for the composite action between the cast-in-place concrete deck and the precast concrete girders. Once the deck is placed, the weight of the plastic concrete causes girder deflection and increases tension in the strands (reduces prestress losses). When the concrete cures, the shrinkage of the hardened concrete deck also compensates for a small portion of the prestress losses. In this study, strand stresses were recorded before and after deck placement to include the effect of the composite girder-deck section in the evaluation of the prestress loss prediction methods.

The accuracy of prestress loss predictions is influenced by the difference between the design concrete properties and the actual concrete properties. Several studies have reported that, for a given girder, the concrete strength measured at the precast concrete plant is higher than the design strength by as much as 60%.[10–12] Fabricators intentionally overdesign the concrete mixture proportions to achieve higher transfer strengths and shorten the production cycle. Higher concrete strengths lead to higher elastic moduli, increasing the stiffness of the girders, decreasing elastic shortening in the concrete, and thus decreasing the prestress losses. The difference between the design and the actual concrete properties also comes from differences in the local concrete materials. Aggregate properties are not consistent among quarries; that is one of the sources of the variation among concrete plants. The properties of the local materials affect the modulus of elasticity, shrinkage, and creep, which in turn affect the prestress losses.[13–16] For example, aggregate porosity, size, shape, and surface texture are different among quarries and precasting plants. These properties affect the strength and stiffness of concrete.[4,17] Denser coarse aggregate leads to concrete with higher strain capacity[18] when there is good bond between the paste and the coarse aggregate. Given that, the prediction of the modulus of elasticity can be improved if the local coarse aggregate is accounted for. Modulus of elasticity determines girder stiffness and is essential for accurate computation of elastic deformations and prestress losses.[3,15,16] In this study, modulus of elasticity predictions were evaluated by testing specimens prepared using several concrete mixture proportions and three different types of coarse aggregate that were used in bridge projects in Arkansas. The properties of these coarse aggregates are shown in **Table 1**. The results of this evaluation were used to develop recommended improvements for predicting modulus of elasticity.

Table 1. Properties of the three types of coarse aggregate

	Sieve	Passing, %		
		Crushed limestone from Sulphur Springs, Ark.	River gravel from Greenwood, Miss.	Crushed limestone from Springdale, Ark.
Sieve analysis	1 in.	100.00	100.00	100.00
	¾ in.	95.15	93.93	97.95
	½ in.	58.97	51.92	60.99
	⅜ in.	43.18	31.73	37.99
	No. 4	6.80	8.27	2.55
	No. 8	0.98	5.33	0.68
	No. 16	0.69	5.33	0.50
Physical properties	Water absorption by mass, %	2.20	2.40	0.70
	Specific gravity	2.58	2.54	2.67
	Particle shape and surface texture	Irregular and rough	Rounded and smooth	Irregular and rough

Note: No. 4 = 4.75 mm; No. 8 = 2.36 mm; No. 16 = 1.18 mm; 1 in. = 25.4 mm.

Production practices also differ among plants and affect the elastic modulus, shrinkage, and creep, changing the strand stress over time.[13,14] Mixture proportioning, concrete strength (at transfer and other ages), curing time, and curing method influence prestress losses. Even storage time and girder support location influence the concrete deformation and prestress losses.[19] Therefore, the most accurate prediction of prestress losses would include laboratory measurements of some design parameters, such as coarse aggregate correction factor, creep coefficients, and the average maximum shrinkage.[3] In this work, extensive concrete material testing was performed at precast concrete plants and at the concrete materials laboratory of the University of Arkansas.

In this study, prestress losses were measured for several full-scale girders and compared with the design values calculated using the approximate estimate and the refined estimates of the 2017 AASHTO LRFD specifications.[6] The approximate estimate is widely used in the design of bridge girders due to its simplicity.[20] This method provides an estimate for the total prestress losses at a final time only. The losses due to creep, shrinkage, and relaxation can be estimated from a single equation using the 2017 AASHTO LRFD specifications Eq. (5.9.3.3-1).

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR} \quad \text{or}$$

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 83\gamma_h\gamma_{st} + \Delta f_{pR} \quad \text{(AASHTO 5.9.3.3-1)}$$

where

Δf_{pLT} = total long-term prestress losses

f_{pi} = prestressing steel stress immediately before transfer

A_{ps} = area of prestressing steel

A_g = gross section area

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

Δf_{pR} = relaxation loss to be taken as 2.4 ksi (16.6 MPa) for low-relaxation strand

The refined estimates method is more accurate and complex than the approximate estimate method. This is because it calculates the losses from each time-dependent source, such as creep, shrinkage, and strand relaxation separately as shown in the 2017 AASHTO LRFD specifications Eq. (5.9.3.4.1-1). The refined estimates method can estimate the losses during any stage of the construction process and for a wide range of prestressed concrete girders with or without a topping slab.

$$\Delta f_{pLT} = \left[\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right]_{id} + \left[\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS}\right]_{df}$$

(AASHTO 5.9.3.4.1-1)

where

$[\Delta f_{pSR}]_{id}$ = prestress losses from transfer to deck placement due to shrinkage

$[\Delta f_{pCR}]_{id}$ = prestress losses from transfer to deck placement due to creep

$[\Delta f_{pR1}]_{id}$ = prestress losses from transfer to deck placement due to relaxation

$[\Delta f_{pSD}]_{df}$ = prestress losses from deck placement to final time due to shrinkage

$[\Delta f_{pCD}]_{df}$ = prestress losses from deck placement to final time due to creep

$[\Delta f_{pR2}]_{df}$ = prestress losses from deck placement to final time due to relaxation

$[\Delta f_{pSS}]_{df}$ = prestress gain due to shrinkage of the deck

Objectives

The main objectives of this study are to improve the prediction of prestress losses by reducing the discrepancy between design and actual concrete properties and assess the performance and the conservatism of the 2017 AASHTO LRFD specifications for prediction of prestress losses. Field measurements included concrete materials testing and internal strain monitoring for several full-scale girders of different sizes and lengths. Through laboratory testing, the coarse aggregate correction factor K_1, creep coefficient, and average maximum shrinkage for concrete materials and mixture proportions used in bridge girders in Arkansas were determined. Four types of girders that are widely used in highway bridges were instrumented (AASHTO Types II, III, IV, and VI girders).

Girder instrumentation

This study was conducted on nine prestressed concrete AASHTO girders that were used in three bridges in Arkansas. The Arkansas Department of Transportation uses standard AASHTO cross sections for most prestressed concrete girders. Currently, these girders are produced at two precast concrete plants, referred to here as plant 1 and plant 2. In this study, the experimental program included instrumenting and monitoring four AASHTO girder sizes: Types II, III, IV, and VI. Two girders were instrumented for each girder type, except for the AASHTO Type VI girders, for which three girders were instrumented. **Table 2** shows some details about

Table 2. Summary of main details about the instrumented girders								
Plant	Girder type	Number of girders	Girder length, ft	Strand diameter, in.	Number of strands	Strand profile	Curing type	Casting date
1	II	2	42	0.5	10	Harped at two points	Steam (16 hours)	12/23/16
1	III	2	63	0.5	26	Straight	Steam (16 hours)	1/11/17
2	IV	2	94	0.5	38	Straight	Steam (18 hours)	2/11/17
1	VI	3	109	0.6	38	Straight	Water (18 hours)	8/17/16

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

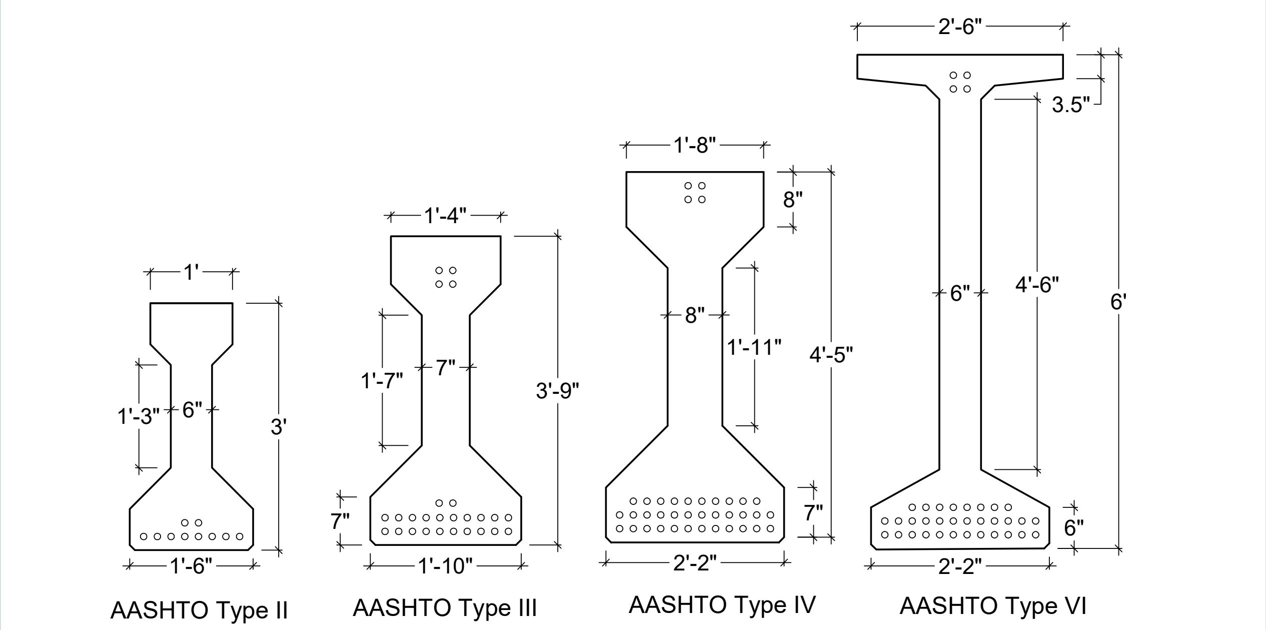

Figure 1. Cross-sectional dimensions for the instrumented American Association of State Highway and Transportation Officials (AASHTO) girders. Note: 1” = 1 in. = 25.4 mm; 1’ = 1 ft = 0.305 m.

the girders included in the study. **Figure 1** shows the cross sections of the girders.

Vibrating-wire strain gauges were embedded in the nine girders to measure the strain. The strain gauges were placed at the midspan of the girder after tensioning the strands and before installing the formwork. One strain gauge was attached to the bottom strands and a second gauge was attached to the top strands or in the center of the top flange when the strands were harped. These gauges also provided temperature readings. Concrete temperature was needed to measure the hydration temperature and thermal gradient along the girders’ height and to correct the strain readings due to the differences in the coefficient of thermal expansion between the steel strands and the concrete. **Figure 2** shows the strain gauges attached to the prestressing strands before the forms were assembled. Strain and concrete temperature were recorded several times before and after placing the concrete; however, the zero readings for the prestress loss measurements were taken just before transfer. After erecting the girders and before placing the deck concrete, the wires of the strain gauges were spliced and attached to the bridge median barrier reinforcement (**Fig. 3**). A manual handheld data reader was used to record the strain.

The girders were cast in different seasons and were therefore subjected to different curing regimens. It is important to point out that none of the girders had an extended curing time, such as over a weekend. This would have significantly reduced

Figure 2. Strain gauges attached to the prestressing strands before installing the side forms.

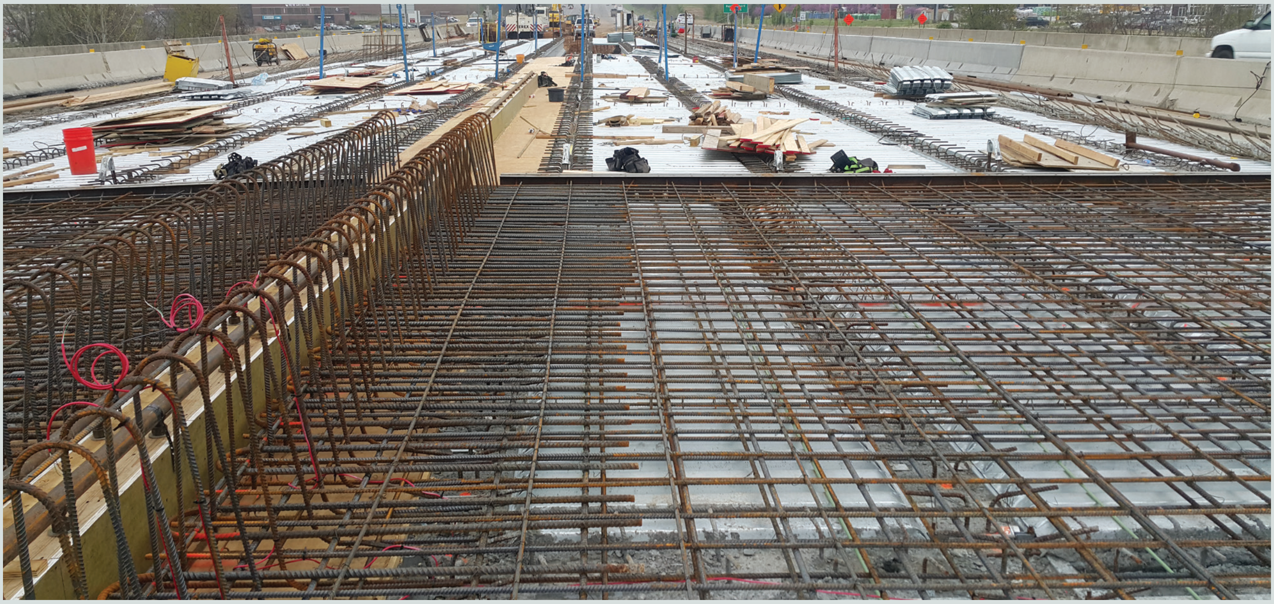

Figure 3. Wires of the strain gauges in the bridge barrier to record the changes in the strands' strain after placing the deck.

the measured prestress losses because the extra curing time increases the elastic modulus of the concrete.[10] Table 2 lists the curing regimen used for each girder. For girders cast in summer, the temperatures measured in the girders exceeded 131°F (55°C), which was enough for the concrete to reach its transfer strength in approximately 16 hours. Steam curing is usually used when the ambient temperature drops below 60°F (15°C) to achieve the required concrete strength earlier and expedite the production process.

Compressive strength and modulus of elasticity testing

Concrete cylinders were prepared during the casting of 21 AASHTO Types II, III, IV, and VI girders (six girders each for Types II, III, and VI, and three girders for Type IV). At least 20 concrete cylinders were cast for each girder size using 4 × 8 in. (100 × 200 mm) plastic molds. Concrete was collected from each girder on the prestressing bed when possible and from at least three different batches. This method was used to make the specimens more representative of the concrete batches that were used to fabricate each girder. The concrete cylinder specimens were placed beside the girders under the curing tarps to mimic the curing conditions of the girders. The compressive strength and the modulus of elasticity of the concrete were tested at transfer and at 28, 56, and 90 days from the time of casting the girders. The tests were needed to calculate prestress losses using the actual concrete properties and compare the results with the design values. At transfer, the compressive strength and the modulus of elasticity were tested using the plant laboratory equipment. At later ages, tests were conducted at the Engineering Research Center at the University of Arkansas. The compressive strength and the modulus of elasticity tests were conducted according to ASTM C39/C39M-18 and ASTM C469/C469M-14, respectively.[21,22]

Comparing the design and the measured concrete properties

Figure 4 compares the design and measured concrete compressive strengths at transfer and at 28 days. In both precast concrete plants, the measured compressive strengths were much greater than the design strength for all girders. At plant 1, the measured compressive strength at transfer exceeded the design values by 73%, 60%, and 27% for AASHTO Types II, III, and VI girders, respectively. For AASHTO Type IV girders that were cast at plant 2, the measured compressive strength at transfer was 59% higher than the design. At 28 days of age, the average measured compressive strength for the four girder types was 69% higher than the design strength.

Because the actual concrete strengths are higher than the design strength, the actual elastic modulus was also higher than anticipated. **Figure 5** compares the measured elastic modulus with the design values found using the 2017 AASHTO LRFD specifications Eq. (C5.4.2.4-2) and the American Concrete Institute's *State-of-the-Art Report on High Strength Concrete* (ACI 363)[23] Eq. (5-1).

$$E_c = 33000 K_1 w_c^{1.5}\sqrt{f'_c} \text{ or } E_c = 0.043 K_1 w_c^{1.5}\sqrt{f'_c}$$

(AASHTO C5.4.2.4-2)

where

E_c = modulus of elasticity of concrete at erection

w_c = unit weight (density) of concrete

f'_c = specified concrete strength at final service conditions

$$E_c = \left[1000 + 1265\sqrt{f'_c}\right]\left(\frac{w_c}{0.145}\right)^{1.5} \text{ or}$$

$$E_c = \left[6900 + 3320\sqrt{f'_c}\right]\left(\frac{w_c}{2320}\right)^{1.5} \qquad \text{(ACI 5-1)}$$

The design modulus of elasticity was calculated using the specified design strength for each girder size, 148.6 lb/ft^3 (2380 kg/m^3) for the unit weight, and correction factor for source of aggregates K_1 coefficient of 1.0. The 148.6 lb/ft^3 unit weight was the average value obtained for several concretes made with the same coarse aggregate used to cast the girders, as discussed later in this paper. The measured modulus of elasticity at the time of transfer exceeded the values expected by the AASHTO LRFD specifications equation by 41%, 30%, 44%, and 15% for the AASHTO Types II, III, IV, and VI girders, respectively.

Shrinkage testing and results

Laboratory tests were performed to determine the shrinkage strain of the concrete used in the girders. During the

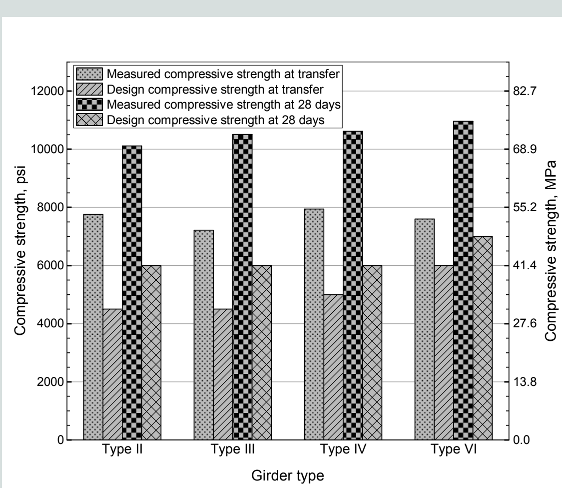

Figure 4. Comparison of the measured and design compressive strengths at prestress transfer and at 28 days.

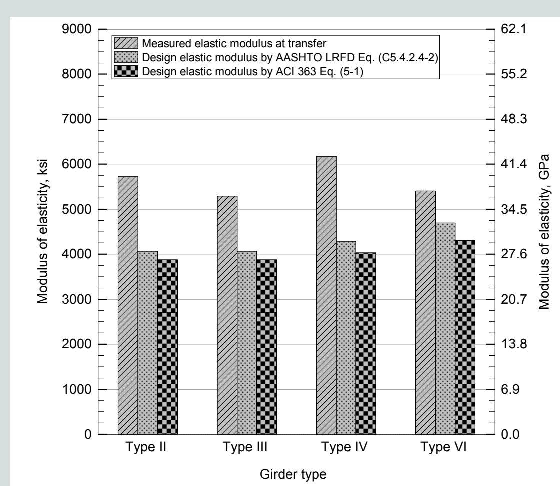

Figure 5. Comparison of the measured and design modulus of elasticity at transfer.

casting of each girder, six concrete prisms were cast and cured beside the girders under the tarps. The prisms were 4 × 4 × 11¼ in. (100 × 100 × 286 mm). After approximately 24 hours, the prisms were shipped to the Engineering Research Center at University of Arkansas and demolded and the initial comparator readings were recorded. Two curing procedures were followed in the shrinkage test. The first procedure followed ASTM C157/C157M.[24] For this curing regimen, three of six prisms were stored in lime-saturated water at 73 ± 3°F (23 ± 1°C). After the 28 days of curing, the prisms were removed from the water storage, wiped with a damp cloth, and measured for the second comparator readings. Following the 28 days of curing, readings for each prism were taken at 4, 7, 14, and 28 days and then monthly for approximately one year.

The second curing procedure did not follow the ASTM method. The remaining three prisms were not submerged in water because this more closely represented the concrete conditions of the girders. After an initial reading, the specimens were stored at a temperature of 73 ± 3°F (23 ± 1°C) and humidity of 50% ± 2%. The comparator readings were taken at 4, 7, 14, and 28 days and then once monthly for approximately a year. Shrinkage strain ε_t was calculated by dividing the change in the prism length by the gauge length using Eq. (1). **Figure 6** shows the test specimens and the length comparator device that was used to measure the shrinkage strain.

$$\varepsilon_t = \frac{L_t - L_{initial}}{L_{gauge}} \tag{1}$$

where

L_t = prism length reading at time t

$L_{initial}$ = initial prism length reading after curing

L_{gauge} = gauge length of 10 in.

The design shrinkage strain is calculated per the 2017 AASHTO LRFD specifications Eq. (5.4.2.3.3-1) by applying correction factors to the maximum basic shrinkage strain. These factors include the effect of concrete strength, ambient humidity, volume-to-surface area ratio, and time.

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} (0.48\times10^{-3}) \quad \text{(AASHTO 5.4.2.3.3-1)}$$

where

ε_{sh} = shrinkage strain

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

k_{hs} = shrinkage correction factor for ambient humidity

k_f = factor for the effect of concrete strength

k_{td} = time development factor

Figure 6. Specimens for shrinkage strain measurements.

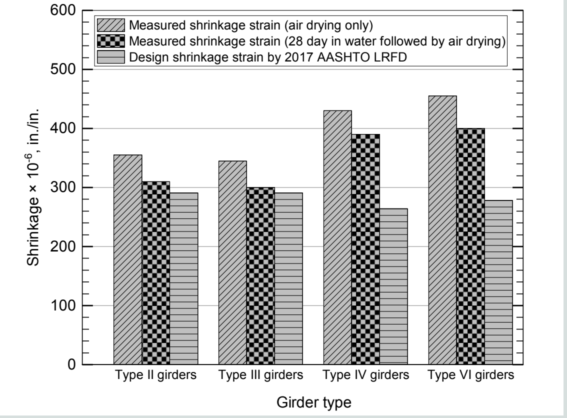

Figure 7. Comparison of the measured and predicted shrinkage strain at 1 year. Note: 1 in. = 25.4 mm.

The equations for the modification factors from the 2017 AASHTO LRFD are as follows:

$$k_s = 1.45 - 0.13\left(\frac{V}{S}\right) \geq 1.0 \text{ or } k_s = 1.45 - 5.117\left(\frac{V}{S}\right) \geq 1.0$$

(AASHTO 5.4.2.3.2-2)

where

V = volume of the concrete member

S = surface area of the concrete member

$$k_{hs} = (2 - 0.014H) \quad \text{(AASHTO 5.4.2.3.3-2)}$$

where

H = average annual ambient relative humidity

$$k_f = \frac{5}{1 + f'_{ci}} \text{ or } k_f = \frac{35}{1 + f'_{ci}} \quad \text{(AASHTO 5.4.2.3.2-4)}$$

where

f'_{ci} = specified concrete strength at transfer

$$k_{td} = \frac{t}{61 - 4f'_c + t} \text{ or } k_{td} = \frac{t}{61 - 0.58 f'_c + t}$$ (AASHTO 5.4.2.3.2-5)

where

t = age of concrete between end of curing and time to consider shrinkage effect or between time of loading and time to consider creep effect for creep calculations

For the prisms sampled from Types II and III girders, the measured shrinkage strains were close to the predicted values calculated using the 2017 AASHTO LRFD specifications (**Fig. 7**). However, for the Types IV and VI girders, the measured shrinkage strain was 48% and 44% greater than the predicted values, respectively. These discrepancies may be attributed to the curing methods. Types II and III girders were heat cured for 13 to 16 hours, which reduced the shrinkage strain, while Types IV and VI girders were not heat cured. It is expected that shrinkage for Types II and III girders would be higher than the predicted values if they had not been steam cured. The concrete for the Types II, III, and VI girders had the same mixture proportions, which eliminates the effect of cement and water content as a possible cause of differences between the shrinkage behaviors of the girders. Therefore, it can be concluded that the AASHTO LRFD specifications equation underestimated concrete shrinkage. Early drying for the prisms that were not water cured increased the shrinkage strain by 13% on average compared with those submerged in water for 28 days. That means specimens subjected to initial wet curing shrunk less and led to measured shrinkage strain closer to the design values.

Creep testing and results

Creep tests were performed on concrete cylinders made with the same mixture proportions and materials used to cast the girders. Creep tests were conducted using 10 concrete cylinders with dimensions of 4 × 8 in. (100 × 200 mm). The cylinder ends were ground and checked for perpendicularity and uniform diameters. Two cylinders were tested for compressive strength, four were kept unloaded to measure the shrinkage strain, and the remaining four cylinders were loaded in the creep frame. Two of the loaded cylinders were sealed with an epoxy sealant to prevent the moisture movement in and out of the concrete cylinders from affecting creep. Two of the unloaded cylinders were also sealed to measure their shrinkage strain.

A steel frame was assembled to apply a constant load on the four creep cylinders, and a hydraulic jack was used to compress four steel springs in the bottom of the frame (**Fig. 8**). The springs were used to create a sustained load on the concrete cylinders after the hydraulic jack was removed. The applied load was 40% of the compressive strength of the concrete cylinders at the time of testing in accordance with ASTM C512/C512M.[25] Reference discs were glued on four locations at 90-degree increments around the circumference of the cylinders before any load was applied. The gauge points were distributed over the cylinders and at least two vertical measurements were taken for each location around the concrete cylinders. Creep strains were obtained using a detachable mechanical strain gauge (Fig. 8). Before loading, two cylinders were tested to determine the compressive strength. Also, before loading, the initial zero readings were taken on each side of all cylinders. Creep coefficient at a specific time can be determined by dividing the creep strain in the concrete cylinders at that time by the elastic strain that accrued immediately after applying the load.

Figure 8. Loaded creep frames and reference discs attached to the concrete cylinders and detachable mechanical strain gauge used in creep strain measurements. Note: DEMEC = detachable mechanical.

The creep strain was monitored for two representative concretes from the two plants. The shrinkage strain was subtracted from the total strain for the sealed and the unsealed specimens. After the initial load was applied, the elastic strain was recorded, and all future strain was assumed to be due to creep and shrinkage. The strain was monitored for one year after the initial loading.

The creep coefficient design values were calculated according to the 2017 AASHTO LRFD specifications using the following equation:

$$\psi(t,t_i) = 1.9\, k_s k_{hc} k_f k_{td} t_i^{-0.118}$$ (AASHTO 5.4.2.3.2-1)

where

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

t_i = concrete age at loading

k_{hc} = humidity factor for creep

$$k_{hc} = 1.56 - 0.008H \qquad \text{(AASHTO 5.4.2.3.2-3)}$$

A few observations can be made from the creep test results. The total strain for the unsealed specimens was 6.2% higher than that for the sealed specimens. The reason is that sealing the cylinders prevents moisture movement out of or into the cylinders. This minimizes the reduction in the volume due to water leaving the concrete. Epoxy sealing also prevented moisture loss and reduced shrinkage. **Table 3** compares the creep coefficients for the sealed and unsealed specimens with the design values calculated according to the 2017 AASHTO LRFD specifications. The measured creep coefficients for the sealed specimens are closer to the design values than the creep coefficients for the unsealed specimens. Unsealed cylinders have a higher creep coefficient because they have higher shrinkage and creep strain.

Table 3. Measured creep coefficients compared with the design values predicted by 2017 AASHTO LRFD specifications

Girder type	Measured (unsealed samples)	Measured (sealed samples)	Predicted
II	1.28	n.d.	1.55
III	1.33	n.d.	1.63
IV	1.31	1.18	1.15
VI	1.22	1.06	1.08

Note: n.d. = no data.

Determining correction factor for source of aggregates K_1

A single equation may not provide a reliable estimate for elastic modulus of concrete with any type of aggregate. Accurate prediction of the elastic modulus requires accounting for the effect of coarse aggregate stiffness through laboratory testing for the compressive strength, unit weight, and elastic modulus.

In the design of prestressed concrete girders, accurate prediction of the elastic modulus is necessary when calculating camber, deflection, and prestress losses.[3,9] The correction factor for source of aggregates K_1 coefficient in section 5.4.2.4 of the 2017 AASHTO LRFD specifications accounts for the effect of coarse aggregate stiffness in the prediction of the concrete's elastic modulus. The modulus of elasticity of concrete determines overall girder stiffness and affects the magnitude of prestress losses, especially the elastic shortening loss, which forms a large part of the total losses.

Three types of coarse aggregates, two crushed limestone and one river gravel, were collected from three different quarries that provide coarse aggregate for girders used in Arkansas. For each type of aggregate, seven concrete mixture proportions were developed with target 28-day compressive strengths ranging from 5.0 to 11.0 ksi (34.5 to 75.8 MPa). The concrete mixture proportions were designed with different amounts of cement, coarse aggregate, and water to obtain correction factor for source of aggregates K_1 values that represent a wide range of concretes. Unit weight, slump, and (for some mixtures) air content were measured for the fresh concrete properties. Compressive strength and modulus of

Table 4. Concrete mixture proportions used for the modulus of elasticity testing specimens

Material	Mixture 1	Mixture 2	Mixture 3	Mixture 4	Mixture 5	Mixture 6	Mixture 7
Cement, lb/yd^3	520	550	564	600	611	658	705
Coarse aggregate, lb/yd^3	1700	1640	1760	1700	1835	1924	1900
Fine aggregate, lb/yd^3	1456	1517	1541	1485	1417	1271	1161
Water, lb/yd^3	276	264	234	270	238	230	282
Air, %	n.d.	n.d.	3.1	1.7	1.5	n.d.	1.8
Water-cement ratio	0.53	0.48	0.415	0.45	0.39	0.35	0.4
High-range water reducing admixture, fl oz/cwt	2	7	5.5	8	8.5	6	7
Slump, in.	4.0	6.5	2.75	7.0	7.0	7.0	10.0
28-day f'_c for limestone from Springdale, Ark., psi	5530	7440	7920	9680	9690	10,513	10,220
28-day f'_c for limestone from Sulphur Springs, Ark., psi	5930	7230	8300	9410	10,000	11,570	10,740
28-day f'_c for river gravel from Greenwood, Miss., psi	4690	7110	7860	8830	8730	9245	9830

Note: f'_c = specified concrete strength at final service conditions; n.d. = no data. 1 in. = 25.4 mm; 1 fl oz/cwt = 0.65 mL/kg; 1 lb/yd^3 = 0.593 kg/m^3; 1 psi = 6.895 kPa.

elasticity were measured at 1, 7, 28, and 56 days of age using 4 × 8 in. (100 × 200 mm) concrete cylinders. At each age, typically, two or three cylinders were tested for compressive strength, and then two or three cylinders were tested for elastic modulus. Compressive strength and modulus of elasticity were tested according to ASTM C39/C39M-18 and ASTM C469/C469M-14, respectively.[21,22] **Table 4** shows details of the mixture proportions and the compressive strength of three concretes with the three types of coarse aggregates.

The 2017 AASHTO LRFD specifications Eq. (C5.4.2.4-2) accounts for the effect of coarse aggregate type by the factor K_1. In National Cooperative Highway Research Program (NCHRP) report 496, Tadros et al.[4] determined the K_1 values for coarse aggregates collected from five different states. The K_1 values, which are the ratio of predicted to measured modulus of elasticity, were 1.037, 1.122, 0.768, and 0.889 for aggregate collected from Nebraska, New Hampshire, Texas, and Washington, respectively. Barr et al.[9] determined a correction factor for source of aggregates K_1 value of 0.896 in a study aimed to calibrate the prediction of prestress losses for the Utah Department of Transportation. In this study, the modulus of elasticity was plotted against the compressive strength using AASHTO LRFD specifications Eq. (C5.4.2.4-2) with the averaged measured unit weight. Then, the correction factor for source of aggregates K_1 coefficient was determined as the value that gives the best-fit curve to the measured modulus of elasticity (**Fig. 9–11**). It is more accurate to find two correction factor for source of aggregates K_1 coefficients with an applicable range, below and above 6500 psi (44.8 MPa), rather than having one correction factor for source of aggregates K_1 value for each type of coarse aggregate. **Table 5** summarizes correction factor for source of aggregates K_1 coefficients that were determined from Fig. 9 through 11.

Predicted and measured elastic shortening loss

The predicted elastic shortening losses were closer to the measured values than the measured total prestress losses were to their predicted values. The design elastic shortening loss was calculated according to the 2017 AASHTO LRFD specifications[6] by applying the prestressing jacking force at the given eccentricity to the transformed section properties. The elastic shortening loss depends mainly on the modulus of elasticity of concrete at the time of transfer. If the concrete elastic modulus is accurately predicted, the elastic shortening loss should be close to the design value. The differences between the design and the measured elastic shortening loss ranged from -10% to 26%; however, when the actual concrete properties were used in the computation, the measured elastic shortening losses were higher than predicted by 2% to 18%. In the authors' opinion, the underestimation in the elastic shortening loss reported here is reasonable and could be due to the measured elastic modulus of the concrete cylinders being higher than the actual stiffness of the girder. **Figure 12** compares the measured elastic shortening loss with the design values.

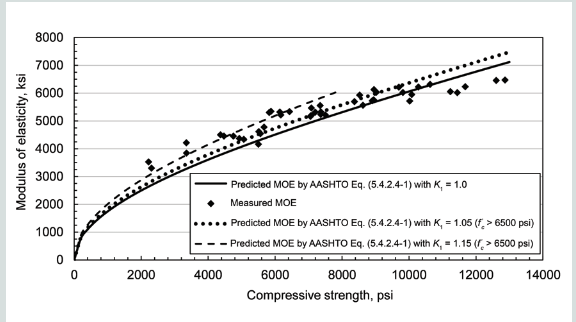

Figure 9. Predicted results compared with experimental results of modulus of elasticity for concrete with crushed limestone from Springdale, Ark. Note: f'_c = specified concrete strength at final service conditions; K_1 = correction factor for source of aggregates; MOE = modulus of elasticity. 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa.

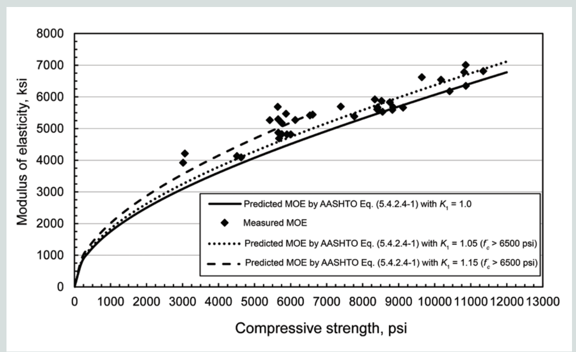

Figure 10. Predicted results compared with experimental results of modulus of elasticity for concrete with crushed limestone from Sulphur Springs, Ark. Note: f'_c = specified concrete strength at final service conditions; K_1 = correction factor for source of aggregates; MOE = modulus of elasticity. 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa.

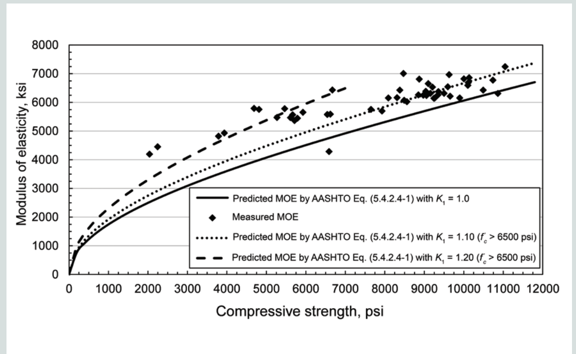

Figure 11. Predicted results compared with experimental results of modulus of elasticity for concrete with river gravel from Greenwood, Miss. Note: f'_c = specified concrete strength at final service conditions; K_1 = correction factor for source of aggregates; MOE = modulus of elasticity. 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa.

Table 5. Correction factor K_1 for each type of aggregate

Range of applicability, ksi	Crushed limestone from Sulphur Springs, Ark.	River gravel from Greenwood, Miss.	Crushed limestone from Springdale, Ark.
$f_c' \leq 6.5$	1.15	1.20	1.15
$f_c' > 6.5$	1.05	1.10	1.05

Note: f_c' = specified concrete strength at final service conditions. 1 ksi = 6.895 MPa.

It is worth mentioning that the friction between the girder ends and the prestressing bed partially restrains the transfer of the prestressing force. A slight increase in the strand strain was observed after moving the girders off the bed. Ward[26] found that the elastic shortening loss that was measured 4.7 hours after transfer was closer to the design values in a study conducted on double-tee prestressed concrete girders.

Predicted and measured total prestress losses

Figures 13 and **14** show the measured total prestress losses and the predicted (or design) prestress losses using the refined estimates method and the approximate estimate method, respectively. For all girders at time of deck placement, the measured prestress losses were less than the predicted values obtained using the refined estimates method in the 2017 AASHTO LRFD specifications.[6] When using the specified design concrete properties in the calculation of the total losses, the percent differences between the measured and the design prestress losses at the time of deck placement are 146%, 86%, 71%, and 14% for AASHTO Types II, III, IV, and VI girders, respectively. Although the deck had not been placed on the Type IV girders at the time, the strand strain recorded at an age of 180 days after casting was assumed to be the strain at deck placement.

One of the reasons for the overestimation of prestress losses is that designers use the minimum specified final concrete strength, while the actual strengths are almost always higher. It was determined that the 2017 AASHTO LRFD specifications' refined estimates method overestimated the total prestress losses at the time of deck placement, especially for the AASHTO Types II and III girders. Camber prediction and prestress loss prediction at any point during construction are directly related to the concrete properties at that age. Also, the predicted camber at the time of deck placement is further complicated because of the differences between measured and predicted prestress losses.

When using the measured concrete properties (concrete elastic modulus, unit weight, time at transfer, and time at deck placement), the refined estimates method predicted losses that were approximately 61% and 33% more than those measured for Types II and III girders, respectively. For the longer girders, the design prestress losses for Types IV and VI girders were 23% and 9% higher than the measured values, respectively. The refined estimates method better predicts prestress losses in Types IV and VI girders than in Types II and III girders. This could be attributed to the sizes of girders that were used to calibrate and verify the prestress loss design equations.[3] Most of the design equations were derived from

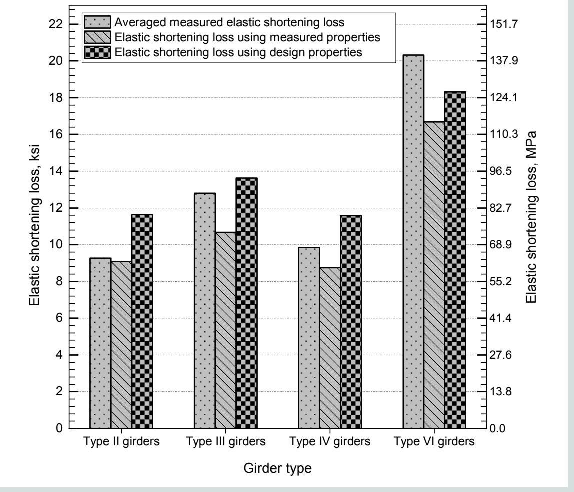

Figure 12. Comparison of the measured elastic shortening losses with expected values calculated using the design properties and the measured properties.

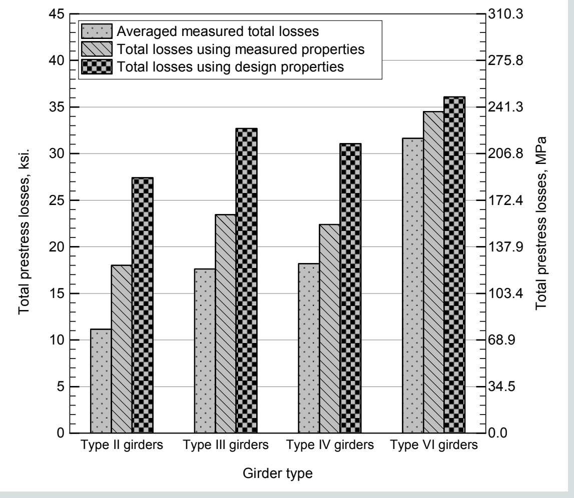

Figure 13. Comparison of the measured total prestress losses with the design values of the 2017 AASHTO LRFD specifications' refined estimates method with that calculated using the design properties and the measured properties of concrete.

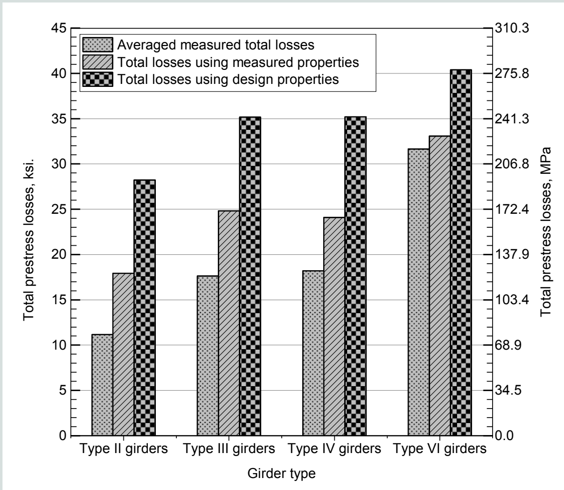

Figure 14. Comparison of the measured total prestress losses with the design values of the 2017 AASHTO LRFD specifications' approximate estimate method with those calculated using the design properties and the measured properties of concrete.

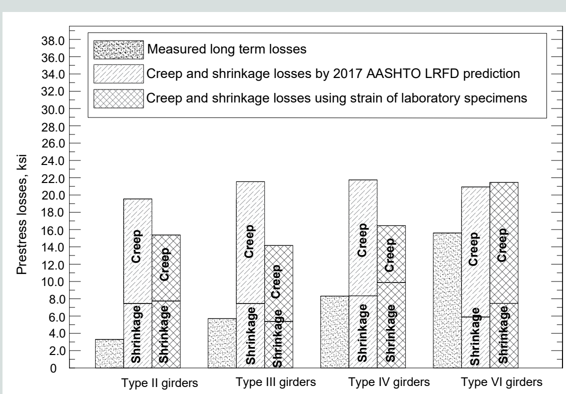

Figure 15. Design creep and shrinkage prestress losses compared with the field and laboratory values.
Note: 1 ksi = 6.895 MPa.

experimental validations on Type VI girders, and that may explain the good agreement between the measured and designed prestress losses for Type VI girders.[3]

Predicted and measured creep and shrinkage losses

To further investigate the differences between the design and the measured prestress losses, it was necessary to compare the shrinkage and creep losses that were calculated based on the laboratory shrinkage and creep specimens with the losses that were recorded from the strain gauges inside the girders. **Figure 15** compares the shrinkage and creep prestress losses that were measured in the girders with the shrinkage and creep losses that were measured from the strain of the prism and cylinder specimens. The design values per the 2017 AASHTO LRFD specifications are also presented. The measured creep and shrinkage losses were determined by subtracting the elastic shortening loss from the total losses measured by the strain gauges. The measured total losses from the strain gauges were taken before deck placements. The shrinkage and creep strains that were measured from the prism and cylinder specimens were converted to prestress losses using the 2017 AASHTO LRFD specifications Eq. (5.9.3.4.2a-1) and (5.9.3.4.2b-1):

$$\Delta f_{pSR} = \varepsilon_{bid} \times E_p \times K_{id} \quad \text{(AASHTO 5.9.3.4.2a-1)}$$

where

Δf_{pSR} = prestress loss due to shrinkage

ε_{bid} = concrete shrinkage strain of girder between the time of transfer and deck placement

E_p = modulus of elasticity of prestressing steel

K_{id} = transformed section coefficient

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} \times f_{cgp} \times \psi(t_d,t_i) \times K_{id} \quad \text{(AASHTO 5.9.3.4.2b-1)}$$

where

Δf_{pCR} = prestress loss due to creep

E_{ci} = modulus of elasticity of concrete at transfer

f_{cgp} = concrete stress at the center of gravity of prestressing strands due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment

$\psi(t_d,t_i)$ = creep coefficient at time of deck placement

The laboratory shrinkage and creep strain should have provided a close estimation of the long-term prestress losses of the girders because the concrete used for both the test specimens and the girders had the same mixture proportions and curing conditions; however, in this study, the laboratory test specimens gave considerably higher losses (Fig. 15). Prestress losses from the test specimens ranged from 5.8 to 12.4 ksi (40.0 to 85.5 MPa) higher than the losses of the strain gauges in the girders. The differences increase when considering the design values. The 2017 AASHTO LRFD specifications' prediction results in higher creep and shrinkage losses by 16.27, 15.84, 13.41, and 5.30 ksi (112.2, 109.2, 92.5, and 36.5 MPa) in Types II, III, IV, and VI girders, respectively. It is clear in Fig. 15 that for Type VI girders, the design equations result in a close estimation to the creep and shrinkage losses from the concrete cylinder and prism specimens. Therefore, it can be concluded from the previous discussion that neither the design equations nor the laboratory creep and shrinkage test specimens gave close estimations of the long-term prestress losses for the AASHTO Types II, III, and IV girders. The following section presents recommendations that can be implemented to decrease the differences between the design and the measured prestress losses.

Outcome of the study

The overestimations in prestress losses were mainly due to the large differences between the actual and design concrete properties (Fig. 4 and 5). It has been confirmed in several precast concrete plants that the actual concrete strength is typically higher than the design strength.[2,12,13,27] The fabricators overdesign the concrete mixture proportions to ensure that the required (design) compressive strength is achieved on time and to avoid delays in the work schedule. When the concrete strength is higher than expected, all of the design parameters that used the compressive strength will be different from the design. Higher concrete strength does not negatively affect the structural capacity of the girders, but the excessive strength should not be neglected—not only when estimating prestress losses but also when estimating camber and deflection of bridge girders.

An effective solution to improve the prediction of prestress losses is to use the anticipated concrete strength at transfer and at 28 days of age rather than using the minimum design strength. A coordinated effort among the owners, the designers, and the precast concrete manufacturers of the local plants to exchange information on similar and previous concrete placements is recommended. Such coordination will help the designer better estimate concrete properties and improve the accuracy of prestress loss, camber, and deflection estimates.

The prestress losses for the girders in this study were recalculated based on the following modifications:

- The compressive strength at transfer was increased by 40%, 30%, 30%, and 10% when the specified design strengths were 4.0, 5.0, 6.0, and 7.0 ksi (27.6, 34.5, 41.4, and 48.3 MPa), respectively.
- The concrete compressive strength at 28 days of age was assumed to be 45% higher than the specified design strength.
- The 2017 AASHTO LRFD specifications Eq. (C5.4.2.4-2) was used to predict the modulus of elasticity with the appropriate correction factor for source of aggregates K_1 from Table 5.

These modifications reduced the average overestimation to 30% compared with 79% when the design concrete properties were used. Implementing such recommendations will result in more efficient design because it will take advantage of the excess concrete strength of the girders. It should be noted that the specified compressive strength in the shop drawings should not be adjusted. **Figure 16** compares the measured and design prestress losses with the predicted losses after implementing the recommendations. The 2017 AASHTO LRFD specifications' refined estimates method and approximate estimate method were used to calculate the total losses to compare with the measured values. This figure shows that the predicted total losses using the recommendations to adjust concrete properties are closer to the measured values than the predicted losses using the design concrete properties calculated using the 2017 AASHTO LRFD specifications.

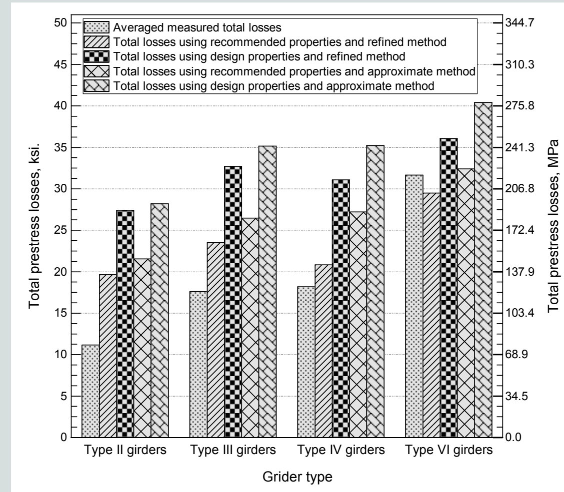

Figure 16. Comparison of the measured prestress losses with the total losses calculated using the expected concrete properties and with the total losses calculated using design concrete properties in the 2017 AASHTO LRFD specifications' refined estimates method and approximate estimate method.

Conclusion

Prestress losses were monitored for AASHTO Types II, III, IV, and VI girders during fabrication, during erection, and shortly after casting the concrete bridge deck. Compressive strength, modulus of elasticity, concrete creep, and concrete shrinkage were measured for all girders and compared with the design values. Based on field and laboratory measurements, the following conclusions were made:

- The measured compressive strength at transfer and at 28 days of age was higher than the design compressive strength by as much as 73%. Precasters use concrete that gives higher compressive strength than that required in the design. This practice is common in the fabrication of prestressed concrete girders in order to achieve the required compressive strength at an earlier time and shorten the production cycle.
- The averaged measured modulus of elasticity at the time of transfer is 32% higher than the expected value calculated using the 2017 AASHTO LRFD specifications Eq. (C5.4.2.4-2).
- The 2017 AASHTO LRFD specifications provide a good estimation for the creep of concrete measured on sealed concrete cylinders.

- For Types IV and VI girders, the measured shrinkage strain was 48% and 44% greater than the 2017 AASHTO LRFD specifications' predicted values, respectively. For Types II and III girders, the measured shrinkage strains were close to the predicted values. Steam-cured girders undergo much less shrinkage than water-cured girders. The 2017 AASHTO LRFD specifications do not account for the curing type in prediction of the concrete shrinkage.

- The measured elastic shortening losses were generally less than those predicted by the 2017 AASHTO LRFD specifications' method because of the higher-than-expected concrete strength. When the actual concrete properties were used in the computation, the measured elastic shortening losses were greater than the predicted losses by 2% to 18%.

- The 2017 AASHTO LRFD specifications' refined estimates method overestimates the total prestress losses for shorter prestressed concrete girders, such as AASHTO Types II and III. When using the measured concrete properties, including the concrete elastic modulus, unit weight, time at transfer, and time at deck placement, the refined estimates method predicts losses that are approximately 61% and 33% more than those measured for Types II and III girders, respectively.

- The measured total prestress losses at the time of deck placement were lower than the design losses calculated using the refined estimates method of the 2017 AASHTO LRFD specifications by 146%, 86%, 71%, and 14% for AASHTO Types II, III, IV, and VI girders, respectively. This overestimation in the total prestress losses can be attributed mainly to the concrete strength being higher than the design strength at transfer and at 28 days of age.

- Using the recommended estimation for the concrete properties improved the prediction of total prestress losses. This will ensure that the design engineer is dealing with more realistic values for stresses in the concrete and prestressing strands.

Acknowledgments

The authors would like to thank the Arkansas Department of Transportation and the Transportation Research Committee for sponsoring this research project. The authors are also very thankful to the two precast concrete plants—Coreslab Structures in Tulsa, Okla., and JJ Ferguson Prestress-Precast in Greenwood, Miss.—for their participation and cooperation in this project.

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Notation

A_g = gross section area

A_{ps} = area of prestressing steel

E_c = modulus of elasticity of concrete at erection

E_{ci} = modulus of elasticity of concrete at transfer

E_p = modulus of elasticity of prestressing steel

= specified concrete strength at final service conditions

f_{cgp} = concrete stress at the center of gravity of prestressing strands due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment

= specified concrete strength at transfer

f_{pi} = prestressing steel stress immediately before transfer

H = average annual ambient relative humidity

k_f = factor to account for the effect of concrete strength

k_{hc} = humidity factor for creep

k_{hs} = shrinkage correction factor for ambient humidity

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

k_{td} = time development factor

K_1 = correction factor for source of aggregates

K_{id} = transformed section coefficient

L_{gauge} = gauge length

$L_{initial}$ = initial prism length after curing

L_t = prism length reading at time t

S = surface area of the concrete member

t = age of concrete between end of curing and time to consider shrinkage effect or between time of loading and time to consider creep effect for creep calculations

t_i = concrete age at loading

V = volume of the concrete member

w_c = unit weight (density) of concrete

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

$[\Delta f_{pCD}]_{df}$ = prestress losses from deck placement to final time due to creep

Δf_{pCR} = prestress loss due to creep

$[\Delta f_{pCR}]_{id}$ = prestress losses from transfer to deck placement due to creep

Δf_{pLT} = total long-term prestress losses

Δf_{pR} = relaxation loss

$[\Delta f_{pR1}]_{id}$ = prestress losses from transfer to deck placement due to relaxation

$[\Delta f_{pR2}]_{df}$ = prestress losses from deck placement to final time due to relaxation

$[\Delta f_{pSD}]_{df}$ = prestress losses from deck placement to final time due to shrinkage

Δf_{pSR} = prestress loss due to shrinkage

$[\Delta f_{pSR}]_{id}$ = prestress losses from transfer to deck placement due to shrinkage

$[\Delta f_{pSS}]_{df}$ = prestress gain due to shrinkage of the deck

ε_{sh} = shrinkage strain

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

$\psi(t_d,t_i)$ = creep coefficient at time of deck placement

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Abstract

Inaccurate prediction of prestress losses leads to inaccurate predictions for camber, deflection, and concrete stresses in a bridge girder. This study aims to improve the prediction of prestress losses and provides bridge designers with insights into the differences between design and actual concrete properties. Prestress losses, compressive strength, modulus of elasticity, shrinkage, and creep were measured for several American Association of State Highway and Transportation Officials (AASHTO) Types II, III, IV, and VI girders. The investigation revealed that the measured total prestress losses at the time of deck placement were lower than the design losses calculated using the refined estimates method of the 2017 *AASHTO LRFD Bridge Design Specifications*. This was mainly attributed to the actual concrete compressive strength at transfer being greater than the design compressive strength. This discrepancy was as high as 73% for some girders. It was also determined that the 2017 AASHTO LRFD specifications' refined estimates method for estimating prestress losses overestimates the total prestress losses at the time of deck placement for AASHTO Types II and III girders.

Keywords

Bridge girder, concrete properties, creep, modulus of elasticity, prestress loss, shrinkage.

Review policy

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

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