EXPERIMENTAL ASSESSMENT OF PRESTRESS LOSSES IN TEXAS BRIDGE GIRDERS

Jose Gallardo, Graduate Research Assistant, The University of Texas at Austin, Austin, TX.
David Garber, Graduate Research Assistant, The University of Texas at Austin, Austin, TX.
Dean Deschenes, Engineering Scientist Associate, The University of Texas at Austin, Austin, TX.
Oguzhan Bayrak, PhD, Professor, The University of Texas at Austin, Austin, TX.

ABSTRACT

In prestressed concrete construction, an initial compressive stress is applied to concrete members to mitigate the potential for cracking under service demands. The loss of prestress is a reduction of the pre-applied stress as a result of concrete strains, such as elastic shortening, creep and shrinkage, and other time dependent phenomena.

Prestress losses within 30 field-representative bridge girders were studied at The University of Texas at Austin. The effects of coarse aggregate type, concrete type, and climate on short- and long-term prestress losses were investigated. The girders were aged for 230 to 980 days under natural climatic conditions at four storage locations. The development of prestress losses was monitored via internal instrumentation on a subset (18 of 30) of the girders. The losses at the end of the experimental program were assessed by testing all the girders in flexure.

The magnitude of the losses correlated well with the inverse of the modulus of elasticity of concrete as influenced by coarse aggregate type and content. The findings of this experimental program can be the basis for the development of improved recommendations for prestress loss estimation.

Keywords: Aggregate, Bridge Girders, Concrete, Prestress Losses, SCC, Stiffness.

INTRODUCTION

Prestressed concrete construction relies on the application of compressive stresses to concrete elements with the objective of reducing the maximum tensile stresses in concrete to prevent such stresses from exceeding the concrete tensile strength. Loss of prestress is the decrease of the effectiveness of this pre-applied stress. The conservative and reliable estimation of prestress losses is important to minimize the potential of crack formation associated with this reduction on the prestressing force. The largest components of the prestress loss are a consequence of concrete deformations: elastic shortening, creep, and shrinkage. The long-term deformations of concrete have been the subject of research for many years. Numerous factors contributing to long-term deformations have been reported in the literature[1, 2, 3]. A compilation of the main parameters affecting creep and shrinkage (in addition to the time factor) can be summarized as follows:

- moisture migration
    - paste parameters (e.g. w/c ratio and degree of hydration)
    - storage conditions (e.g curing and ambient relative humidity)
    - member geometry
- stress level
    - magnitude of stress (e.g. prestressing and external loads effects)
    - strength of concrete
- stiffness (restraint against deformation)
    - aggregate stiffness and content
    - reinforcement

In this study, 30 field-representative bridge girders were built and conditioned under various climatic conditions; outdoor storage of the girders at four distinct locations (up to 330 miles apart) provided a realistic representation of climatic conditions experienced in the field. Long-term monitoring of the prestress losses within 18 girders was conducted by measuring the changes in concrete strains over time. Flexural testing was conducted on all specimens as a method to assess the total prestress losses at the end of the experimental program period. An evaluation of the concrete material properties was conducted in parallel with the girder program. Collectively, the results of the long-term monitoring, flexural testing and material testing, were used to study the influence of parameters such as: concrete stiffness, storage conditions, and time, on the prestress losses. Analysis of the experimental program data provided insights that guided simplifications[5] of the prestress loss estimation methods found within AASHTO4 .

EXPERIMENTAL PROGRAM

A total of 30 specimens were fabricated, conditioned and tested during the course of the experimental program. At the time of final prestress loss assessment, the four series: I, II, III and IV were representative of a broad range of concrete materials, precast fabrication processes, and climatic conditions encountered in the State of Texas. Limestone or river gravel was utilized as coarse aggregate in conventional and self-consolidating concrete mixtures.

The girders were 45.5-ft long with Type C (conventional AASHTO girder) and Tx46 (bulb-T) cross-sections. The amount of prestressing steel in each specimen varied from about 1.13 percent to 1.17 percent of the gross cross-sectional area. An initial tensile stress of 202.5 ksi was applied to the strands during jacking. This large amount of prestressing steel, for this short span, was used in order to generate high initial compressive stresses, on the order of $0.65f'_{ci}$, to maximize the potential for prestress losses. The girders were fabricated at three different precast plants, and stored at four locations. The experimental matrix, including variation of the test parameters and adopted nomenclature, is presented in Table 1. The conditioning for the long-term development of prestress losses was conducted in 4 locations: Austin, San Antonio and Elm Mott with relative humidity of approximately 62%, and Lubbock (RH≈ 51%). The storage site in Lubbock is shown in Figure 1.

Table 1. Experimental matrix

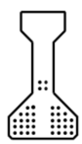
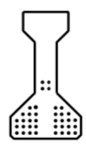
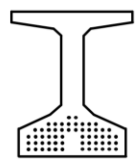
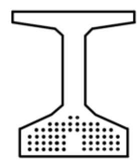
	Series I	**Series II**	**Series III**	**Series IV**
Cross-section & # of ½" low-relaxation strands	Type C  38 strands	Type C  38 strands	Tx46  58 strands	Tx46  56 strands
Coarse Aggregate	Limestone 	River Gravel 	Limestone 	River Gravel 
Storage Location (# of girders)	San Antonio (2) Austin (3) Lubbock (3)	Elm Mott (2) Austin (3) Lubbock (3)	San Antonio (2) Austin (3) Lubbock (3)	Austin (6) (3: SCC + 3: CC)

Figure 1 One of the four storage sites (Lubbock, Texas)

PRESTRESS LOSS ASSESSMENT: STRAIN MONITORING

The monitoring of strains in the strands allows the independent assessment of the prestress losses resulting from elastic shortening and long-term creep/shrinkage. Three to four vibrating wire gages (VWGs) were installed in 18 of the 30 specimens (stored in Lubbock and Austin, Texas) to measure the strains in the concrete along the longitudinal axis of the specimen. The gages were located at various heights through the depth of the girder cross-section, as shown in Figure 2. The gages were attached to a strand when the gage location was close to a strand, otherwise the gage were installed on an auxiliary reinforcement bar supported on the mild reinforcement. Due to compatibility between the prestressing strands and the surrounding concrete, it is possible to use measured strains in the concrete to calculate the average strain in the strands. The strain at the centroid of the strands was obtained from the measured concrete strains through interpolation as shown in Figure 3. The strain at the strands' centroid (ε_p) can be directly related to the prestress losses (Δf_p) by Equation 1.

Figure 2 Installation of vibrating wire gages

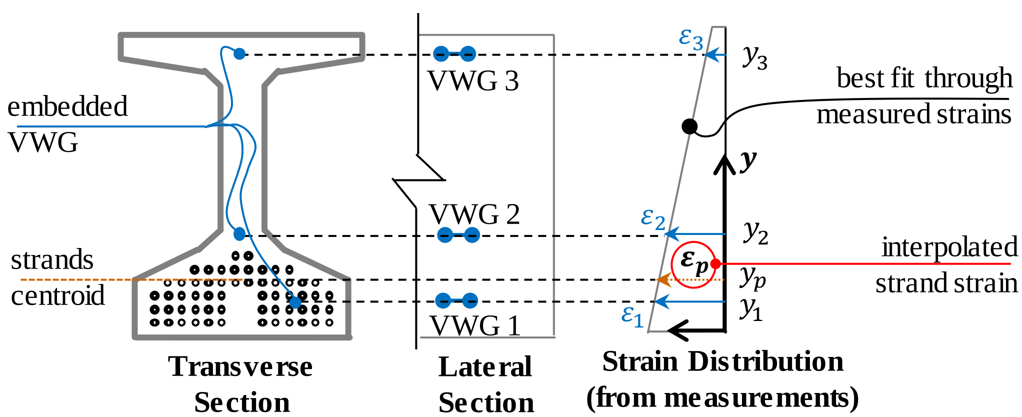


Figure 3 Interpolation of measured strains to obtain the strain at the strand centroid

$$\Delta f_p = \varepsilon_p \cdot E_p + \Delta f_{p\_Relaxation}$$ Equation 1

where,

Δf_p	= total prestress loss (ksi)
ε_p	= strains at the centroid of the strands (in./in.)
E_p	= modulus of elasticity of the strands (ksi)
$\Delta f_{p\_Relaxation}$	= loss of prestress due to strand relaxation (ksi)

Only the strain-related components of the losses ($\varepsilon_p \cdot E_p$) are measurable through strain monitoring; the relaxation loss ($\Delta f_{p\_Relaxation}$) is not. This last type of losses is the time-dependent decrease in stress in the strand that occurs under constant strain; therefore it cannot be quantified based on strain measurements in an element that is simultaneously experiencing inelastic time-dependent deformations. The relaxation losses were estimated through the use of the AASHTO[4] recommendations (Equation 2). The calculated relaxation loss was then added to the strain-related losses to determine the total losses. The estimation of strand relaxation introduces little error because the relaxation associated with low-relaxation strands represents a small fraction of the total losses[6].

$$\Delta f_{p\_Relaxation} = 2 \cdot \frac{f_{pt}}{30} \quad \frac{f_{pt}}{243\ ksi} - 0.55$$ Equation 2 (modified from AASHTO[4] 5.9.5.4.2c-1)

where,

f_{pt}	= stress in the strands after transfer (ksi)

PRESTRESS LOSS ASSESSMENT: FLEXURAL CRACKING TEST

The cracking moment obtained from flexural testing can be used to back-calculate prestress losses using Equation 3. The flexural cracking test provides a comprehensive assessment of all types of prestress losses, whether related to internal or external factors, or short or long-term phenomena. A limitation of the flexural testing is that it only allows the assessment of losses at a single point in time. Flexural testing was used in this study to assess the final total prestress losses (at the end of the conditioning time) on all 30 girders. The four-point load setup used to conduct flexural testing is shown in Figure 4.

$$\Delta f_p = f_{p\_jack} - \frac{M_g \dfrac{y_{t\_c}}{I_c} - f_{ct\_cr} + M_{Pcr} \dfrac{y_{t\_tr}}{I_{tr}}}{A_{ps} \left(\dfrac{1}{A_c} + \dfrac{e_{p\_c} \cdot y_{t\_c}}{I_c} \right)}$$ Equation 3

where,

M_g = moment due to self-weight at studied section (midspan) (kip-in.)
M_{Pcr} = moment due to cracking load at studied section (midspan) (kip-in.)
y_{t_c} = distance from extreme tension fiber to centroid of concrete section (in.)
y_{t_tr} = distance from extreme tension fiber to centroid of transformed section (in.)
I_c, I_{tr} = moment of inertia of the concrete and transformed section, respectively (in.^{4})
f_{p_jack} = average stress in the strands at the end of jacking (ksi)
A_{ps}, A_c = total sectional area of strands and area of concrete section, respectively (in.^{2})
e_{p_c} = distance from centroid of concrete section to centroid of the strands (in.)
f_{ct_cr} = tensile stress in the bottom fiber right before cracking load (ksi).

The first flexural cracking was identified by: (1) visual inspection and (2) load-deflection analysis. Of the two methods, visual detection of first flexural cracking resulted in prestress loss assessments of more significant variability. The increased variability is attributed to a number of factors.

- Premature cracking: Small cracks may appear in localized areas of the beam fascia that are subject to higher stresses and/or lower concrete tensile strengths. These cracks may not be representative of the average response of the specimen, but are still subject to identification via visual inspection.
- Human error: Detection of the first crack relied on the ability of the researchers to perceive the crack, which is highly dependent on the individual and circumstances under which the visual inspections were conducted. Examination of the full surface of the bottom flange with a microscope was not practical, though it may have led to more repeatable identification of first flexural cracking.
- Surface condition: Detection of the earliest flexural cracks was also influenced by the condition of the concrete (shrinkage cracks, concrete surface roughness, color uniformity, etc.). In some cases the beams had rough surfaces and were significantly rust stained from conditioning; this made visual crack detection much more difficult.

A consistent, less variable, method for identification of first flexural cracking was therefore developed on the basis of the measured load-deflection response. Assessments made on the basis of the load-deflection measurements were indicative of the global response of the specimen and were not influenced by the variation of the specimen condition, material properties or researcher capabilities. The procedure for determining the first cracking load on the basis of the load-deflection response is illustrated in Figure 5. This procedure includes:

- Discretization of response: The data from the load-deflection response was discretized into displacement steps of 0.02 inches. This discretization allowed for the stiffness of the response to be consistently calculated between each step and from specimen to specimen.
- Calculation of stiffness: The stiffness was calculated as Δforce/Δdisplacement for each displacement step. During the inspection stages (on which the loading was suspended) creep of the specimen resulted in the loss of load at a constant displacement. These drops were not related to crack occurrence and the stiffness was not calculated for the steps that coincided with inspection stages.
- Calculation of moving average: The moving average of the stiffness was calculated and plotted. These represented the average behavior of the beam from the beginning of the test to the beginning of each of the loading steps.
- Identification of stiffness drop: At the beginning of each test, the flexural stiffness varied within a well-defined band centered about the moving average; behavior that was indicative of an uncracked flexural response. Cracking was therefore noted to occur when the flexural stiffness consistently fell below the moving average; cracks reduced the effective area (and consequently the inertia) of the concrete that resists the flexural demands. This point marked the end of the linear behavior and was identified as first flexural cracking.
- Determination of first cracking load: The deflection corresponding to first flexural cracking was utilized within the context of the load-deflection response to obtain the flexural cracking load.

There is uncertainty in the cracking load value exists even with the use of this method, mainly because the crack initiation and growth was a very subtle process in many of the specimens.

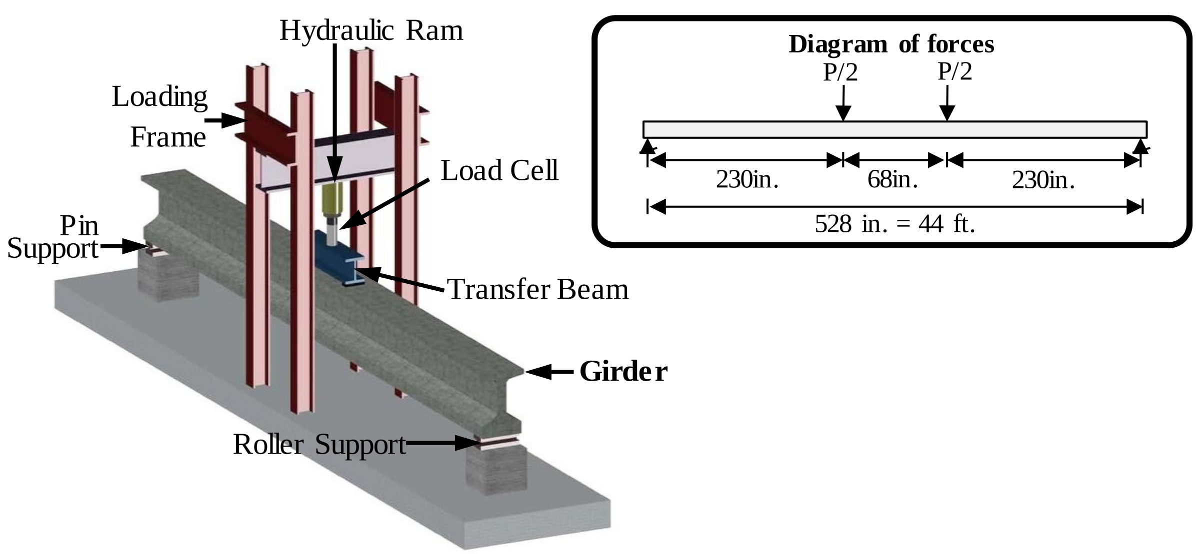


Figure 4 Flexural test setup

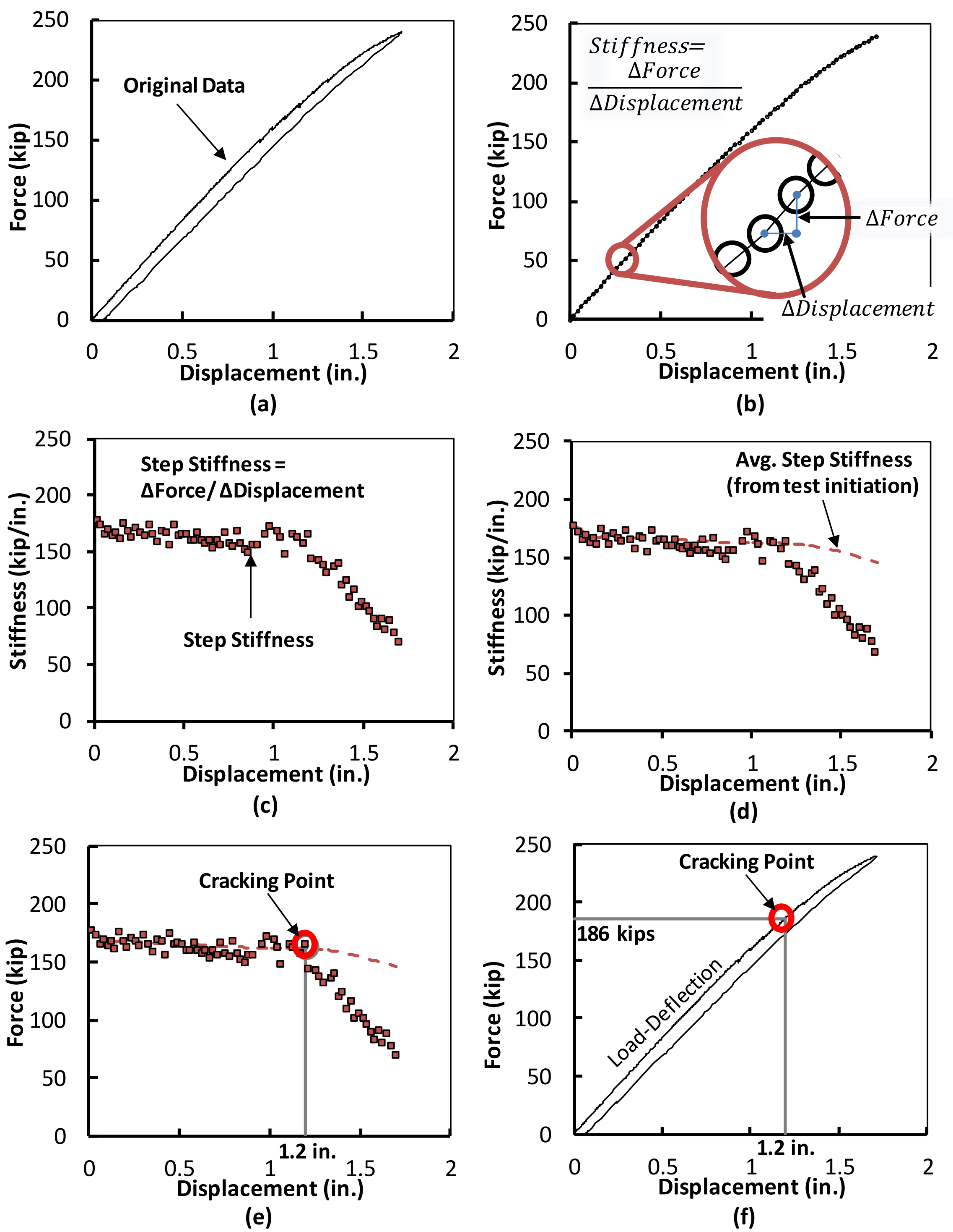


Figure 5 Load-deflection analysis: (a) load-deflection response, (b) discretization of response, (c) calculation of stiffness, (d) calculation of moving average, (e) identification of stiffness drop, (f) determination of first cracking load.

CONCRETE MATERIAL TESTING: MECHANICAL PROPERTIES

The compressive strength, tensile strength, and stiffness of concrete were measured through testing of companion, moist cured, 4-inch cylinders in accordance with ASTM standards C 39/C 39 M, C 496/C 496M, and C 469 respectively. The compressive strength at release, 28 days, and at the time of testing (or final age), and the tensile strength of concrete at time of testing (or final age) are shown in Table 2.

Table 2 Summary of concrete strength

Series	At Release		28 days	Final	
	Age (days)	f_{ci} (ksi)	f'_c (ksi)	$f_{c,\,final}$ (ksi)	$f_{ct,\,final}$ (ksi)
I	1.08	7.0	10.7	10.6	0.83
II	0.98	6.6	11.6	12.7	1.00
III	1.77	6.6	9.6	11.8	0.91
IV-SCC	0.74	6.3	11.5	15.0	1.06
IV-CC	0.74	6.9	12.0	14.1	1.06

The tensile strength of concrete is necessary to back-calculate the prestress loss from the results of the flexural testing assessment of prestress losses (Equation 3). Split cylinder tests were conducted at the time of testing to assess the tensile capacity of the concrete. The split cylinder test was considered as the most reliable standard test to assess the tensile strength for large scale girders given the poor scaling of the modulus of rupture test.

The stiffness of the concrete at time of release (E_{ci}) was measured because it strongly influences the estimation of prestress losses. For design purposes, the initial stiffness of concrete is calculated on the basis of the prescribed strength of the concrete at time of release (f_{ci}) and can be estimated using Equation 4. If the concrete unit weight is 0.145 kips per cubic foot and the K_1 parameter is taken as 1.0, then Equation 5 can be used.

$$E_{ci} = 33{,}000\ K_1 w_c^{1.5}\ \sqrt{f_{ci}}$$

Equation 4
Adapted from AASHTO[4] (5.4.2.4-1)

$$E_{ci} = 1820\ \sqrt{f_{ci}}$$

Equation 5
(for w_c = 0.145 kcf and K_1=1)

where,

E_{ci} = modulus of elasticity of concrete at time of release (ksi)
K_1 = correction factor for source of aggregate
w_c = unit weight of concrete (kcf)

The concrete stiffness associated with each series of specimens in this study was affected by the type of coarse aggregate (see Figure 6). The measured modulus for concrete made using river gravel coarse aggregate is higher than that for concrete made using limestone; while the variation in the initial compressive strength within all series was small (see Table 2).

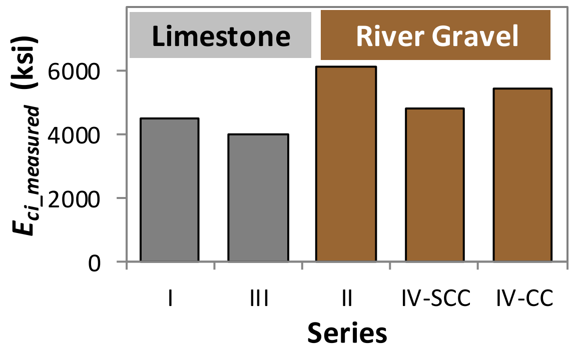


Figure 6 Measured modulus of elasticity per series

PRESTRESS LOSSES RESULTS

Final prestress losses obtained from the long-term strain monitoring measurements are shown in Figure 7, and losses back-calculated from the flexural testing are shown in Figure 8.

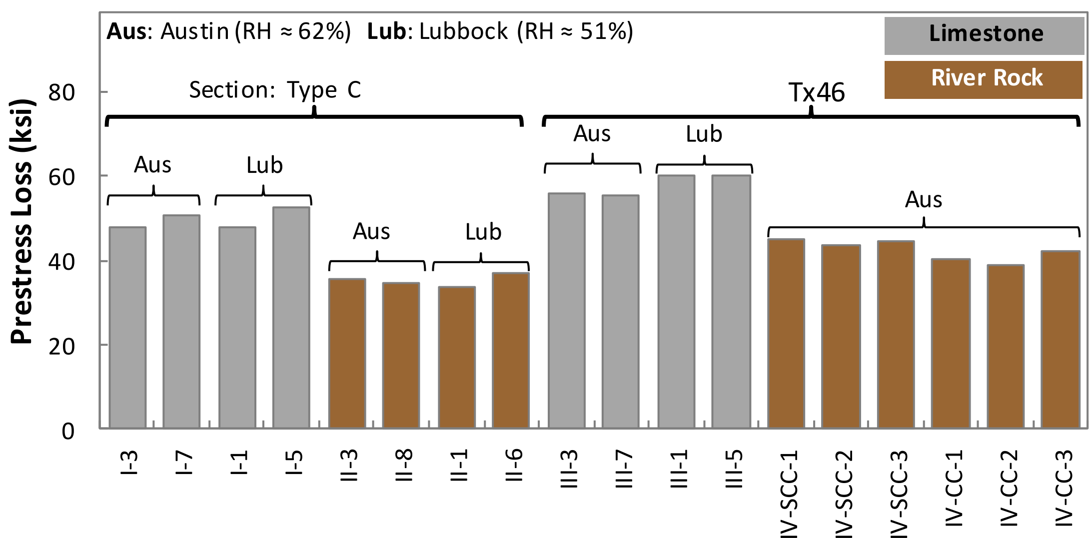


Figure 7 Prestress loss results from the long-term strain monitoring

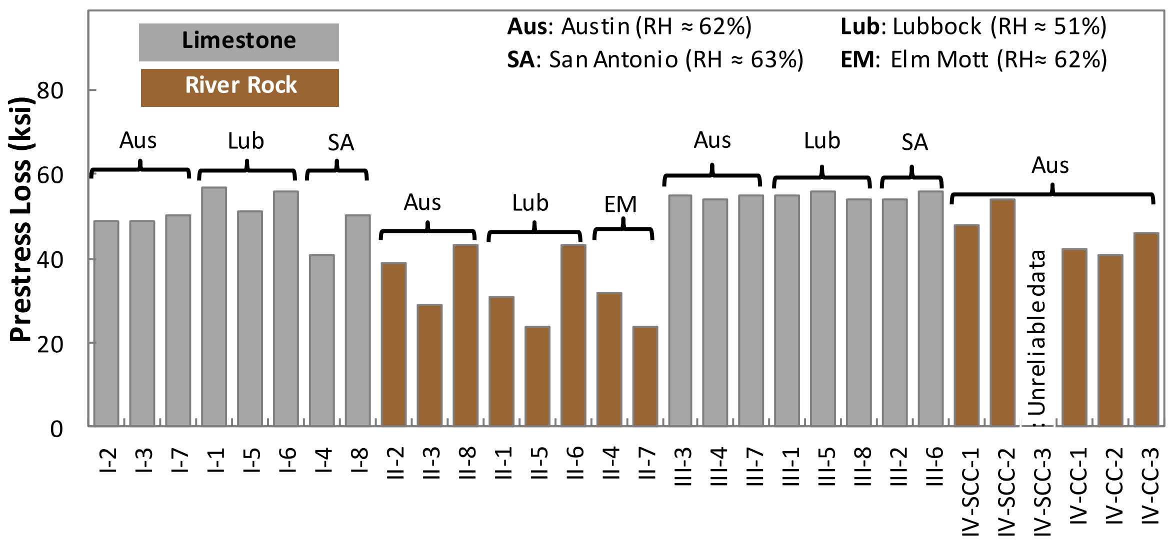


Figure 8 Prestress loss results from flexural testing

In Figure 9, the averages of prestress losses obtained from stain monitoring are compared with averages of prestress losses back-calculated from the flexural testing. Good agreement is observed within the averages obtained from both methods, although there is larger variability observed in the flexural testing results. The variability of the results from the flexural testing method is mainly related to the uncertainty in the parameters required to back-calculate the prestress losses using Equation 3; specially relevant are, the already mentioned uncertainty in the cracking load value, and the uncertainty in the value of the average concrete stress at cracking (estimated from the split-cylinder results) .

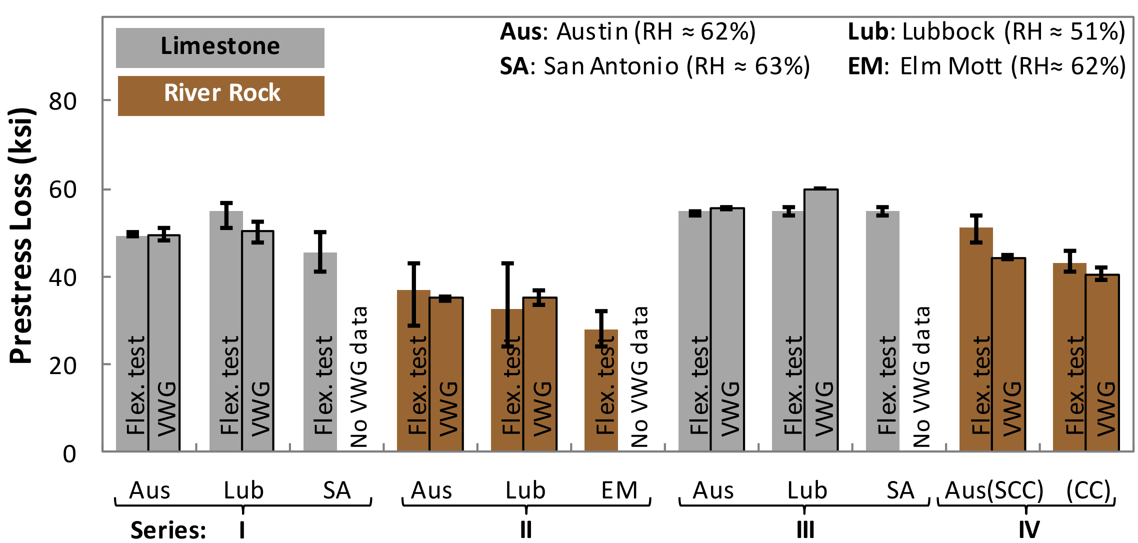


Figure 9 Average prestress loss per group of specimens (strain monitoring and flexural testing results)

EFFECT OF RELATIVE HUMIDITY

Within each series, a portion of the specimens were conditioned in Lubbock, with an average annual ambient relative humidity of approximately 51 percent, and a portion in Austin, with an average relative humidity of about 62 percent. This was done in order to investigate the influence of climate on the development of prestress losses.

The time dependent variation of prestress loss in Series III specimens conditioned in Lubbock and Austin are shown in Figure 10. It can be seen that the elastic shortening loss in both sets of specimens is identical, as was expected. The long-term, strain-related loss (Δf_{p_s}) was slightly larger in the specimens conditioned in Lubbock versus those conditioned in Austin, 58 and 53 ksi respectively; a 10 percent decrease in the relative humidity resulted in an increase of almost 18 percent in the long-term loss within otherwise identical specimens. The prestress loss increase attained through conditioning in a lower humidity environment is consistent with the trends for the concrete creep and shrinkage suggested by the ACI[7] and included in Article 5.4.2.3 of AASHTO[4]. It should be noted that comparison of the identical specimens within Series I and II did not reveal any significant effect of the conditioning environment; this trend can be partially related to the fact that specimens for Series I and II were relocated in the storage locations at later ages (51 and 22 days respectively) than Series III (18 days).

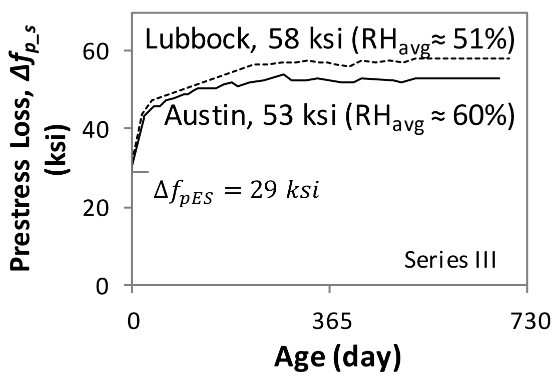


Figure 10 Influence of average relative humidity on time-dependent prestress losses for Series III

TIME DEPENDENCY OF PRESTRESS LOSSES

The time-dependent variation of strain-related prestress losses (Δf_{p_s}) for all internally instrumented specimens is shown in Figure 11. The normalized time-dependent development of the prestress losses within all of the specimens that conditioned for one year or longer is plotted in Figure 12. The prestress loss in a given specimen at a given point in time is normalized by the prestress loss measured at one year. The prestress loss measured at one year was selected as it is a fair representation of the full-term losses measured during the course of the study; prestress losses after 3 years are only about 10 percent larger than those measured at the end of the first year. The most notable aspect of Figure 12 is that about 90 percent of the one-year prestress losses generally occurred within the first four months following transfer of the prestressing force. In general, it is expected that after four months from fabrication, the long-term behavior of the prestress concrete bridge girders will show little changes until the deck is cast or the support conditions are changed.

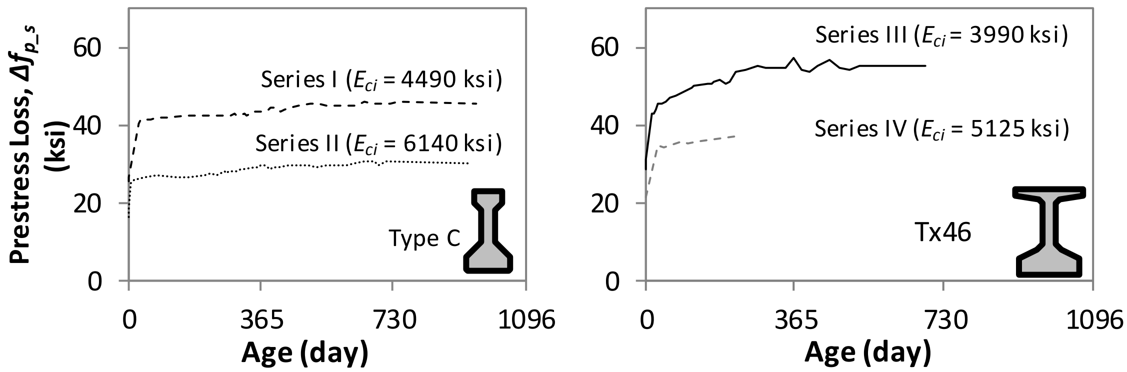


Figure 11 Time development of average prestress losses per series

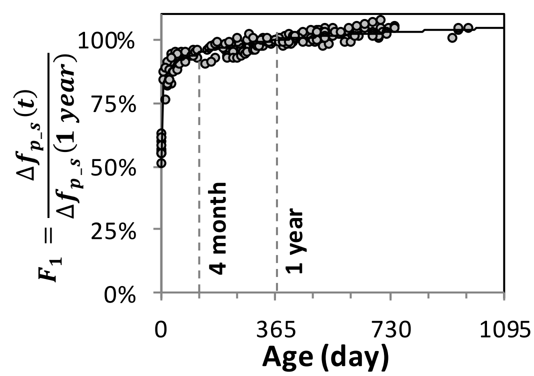


Figure 12 Normalized time-dependent development of prestress losses

RELEVANCE OF CONCRETE STIFFNESS

The stiffness of the specimen as a whole restrains a portion of the long-term deformations of concrete; as suggested in the literature[1]. This statement is applicable at different levels:

> (1) Local: The coarse aggregate stiffness and content has a strong effect on the long-term deformations of concrete; as proposed in the literature: *"…aggregate content and modulus of elasticity are the most important parameters affecting creep*"[2], and: "*The most important influence is exerted by aggregate, which restrains the amount of shrinkage…*"[3]
>
> (2) Global: The concrete stiffness significantly restrains the elastic shortening, creep and shrinkage deformations that are the main source of prestress losses

Both effects are interrelated as observed within this study; beams made with stiffer concrete (e.g. those made using river gravel), experienced smaller prestress loss than beams made with softer concrete (e.g. those made using limestone).

The average final prestress loss for all the series is shown in Figure 13**Error! Reference source not found.**. The total loss is broken into elastic shortening and long-term loss components to help illustrate the effect of the stiffness on each. The series of specimens constructed with stiffer, river gravel concrete experienced significantly smaller total prestress loss: Series I and III experienced total losses of 50 ksi and 58 ksi on average, respectively, while Series II, Series IV-SCC and Series IV-CC only experienced total losses of 33 ksi, 43 ksi and 39 ksi, respectively.

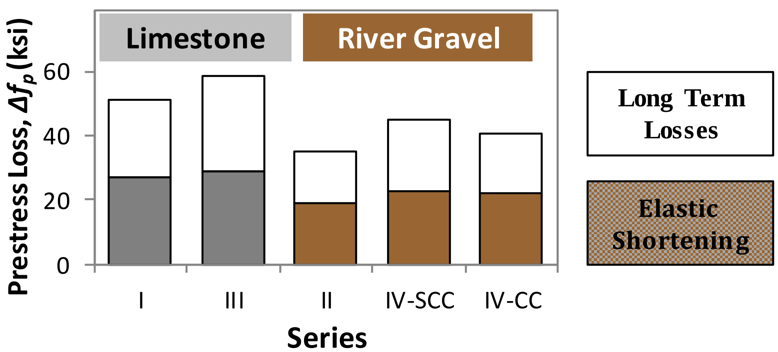


Figure 13 Average prestress losses per series, based on strain monitoring.

The concrete stiffness also influenced the long-term losses greater than elastic shortening. Within Series I and II, the elastic shortening loss decreased by 30 percent and the long-term loss by 40 percent when a stiffer concrete was used. Within Series III and IV, the elastic shortening loss decreased by 20 percent and the long-term loss by 36 percent when a stiffer concrete was used. These observations are consistent with common assertions that creep and shrinkage are heavily influenced by the coarse aggregate properties[7]. The correlation between the strain-related prestress losses (Δf_{p_s}) and the modulus of elasticity is clear within the four series, as shown in Figure 14. The correlation between the prestress losses and the elastic shortening is considered as a more general correlation because it involves the effect not only of the concrete stiffness, but also of the initial stress in the concrete at the centroid of the prestressing strands (Δf_{cgp}); which can be estimated using Equation 6. A strong correlation exists within the elastic shortening and the total prestress loss (see Figure 15).

$$\Delta f_{pES} = \frac{f_{cgp}}{E_{ci}} E_p \qquad \text{Equation 6}$$

where,

E_{ci} = modulus of elasticity of concrete at time of release (ksi)
E_p = modulus of prestressing tendons (ksi)

f_{cgp} = concrete stress at center of gravity of prestressing steel at transfer (ksi)

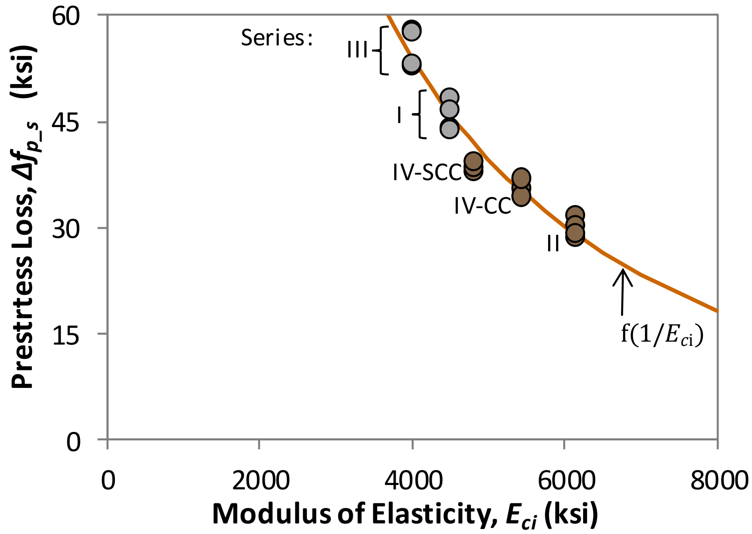


Figure 14 Correlation between the prestress losses and the measured modulus of elasticity

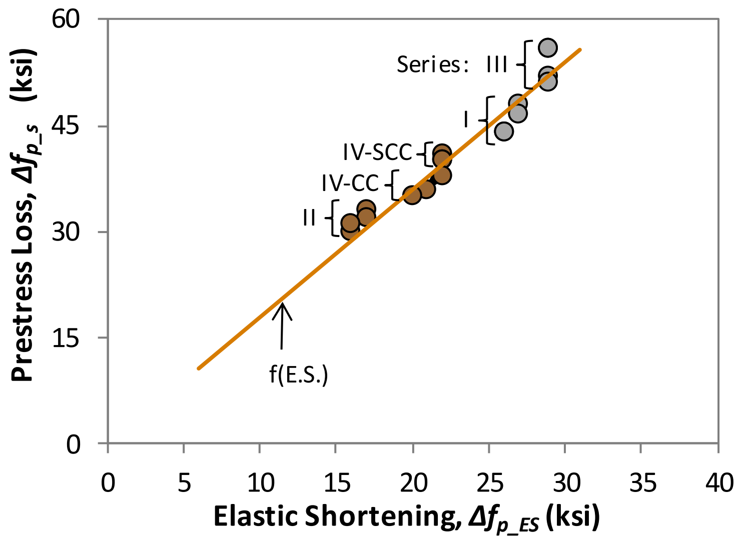


Figure 15 Correlation between the prestress losses and the measured elastic shortening

THE K1 FACTOR

The AASHTO[4] prestress loss provisions accounts for the variation in concrete stiffness (as a result of constituent properties and/or mixture proportions) by the use of the K_1 factor, as introduced in Equation 4 above. Given measurements of the concrete modulus and compressive strength, K_1 can be calculated as the ratio of the measured and estimated concrete moduli. Using **Error! Reference source not found.**, K_{1_test} can be calculated on the basis of the measured compressive strength. The values obtained for this study are presented in **Error! Reference source not found.**. The K_1 factor for each of the concrete mixtures used in the current project varied between 0.87 and 1.20.

$$K_{1\_test} = \frac{E_{ci\_measured}}{1820 \sqrt{f_{ci}}} \qquad \text{Equation 7}$$

Table 3. K_{1_test} values for the concrete mixtures used

Series	K_{1_test}
I	0.91
II	1.2
III	0.87
IV-SCC	1.0
IV-CC	1.15

The conservatism of the estimated modulus of elasticity was considered by Tadros et al.[8], who supported the use of a factor (K2) for the estimation of lower-bound modulus of elasticity "*appropriate for prestress loss*"; the average was found to be $K_{2\_(10th\ percentile\_all\ data)} = 0.777$. The method proposed by Tadros et al[8] to estimate prestress losses, with few modifications, is contained in AASHTO[4]; however, the K_2 factor is not included.

The current language in AASHTO[8] allows for K_1 to be taken as 1.0 if material testing was not conducted. A bridge designer generally does not know which fabricator or what type of aggregates will be used for a given structure until the design is complete and the bridge has been let for construction. They will likely use the default K_1 value of 1.0. Moreover, it is likely that such an approach will result in unconservative estimates of prestress loss, especially for pretensioned girders fabricated with soft coarse aggregates. In theory, if a concrete for which $K_{1\_test} = 0.8$ is used to fabricate a girder designed using K_1=1.0 together with an ideal method (perfectly accurate and precise), the losses will be underestimated by approximately 20 percent.

For typical designs in which conservatism and simplicity are desirable, it is recommended that a conservative value of K_1 (lower bound of K_{1_test} values for the range of mix designs that are used in the field, or $K_1 \cdot K_2$ if using Tadros nomenclature) be used. A more accurate K_1 value may be used if either (1) material testing is conducted or (2) there is accepted knowledge that the mix design that will be used produces concrete with adequate stiffness.

If a higher precision in the estimation of the prestress loss is desired for specialty structures, it is recommended that modulus of elasticity be specified (in addition to the compressive strength) when construction drawings are submitted. Currently, the use of elaborate estimation methods has not been matched with requirements of verifying the mechanical properties (other than compressive strength) of the concrete used during fabrication. Considering the large variability observed in concrete properties, it is difficult to achieve high precisions without specifying the required value of the most relevant parameter on the estimation of prestress losses: concrete stiffness.

SUMMARY AND RECOMMENDATIONS

Results from the assessment of concrete material properties and prestress losses within 30 full-scale, field-representative girders may be summarized as follows:

1. The total losses showed a good correlation with the inverse of the concrete stiffness.
2. Concrete made with river gravel concrete was stiffer than that made with limestone. Furthermore, conventional concrete was marginally stiffer than, an otherwise similar, self-consolidating concrete.
3. The influence of average relative humidity on the long-term prestress losses was marginal (less than 10%, for a variation of approximately 10% on the relative humidity).

Based on the study, the following can be recommended:

1. Emphasize the effect of the modulus of elasticity of concrete on the estimation of the total losses when designing pretensioned girders,
2. Use conservative estimations of the modulus of elasticity according to the anticipated range of mix designs used in practice. This can be achieved through the use of conservative K_1 values (in the AASHTO[4] equations) to estimate the modulus of elasticity based on concrete strength (the minimum valued observed in this study was of K_1=0.87).
3. For cases in which the accuracy of prestress loss estimation is critical, as may be the case for specialty structures, the use of an accurate modulus of elasticity is recommended, which can be achieved through testing; in §C5.4.2.4 AASHTO[4] it is stated that the "*use of a measured K_1 factor permits a more accurate prediction of modulus of elasticity and other values that utilize it*"[4].
4. Analyze data from previous research and verify if the relationship between total prestress losses and the elastic shortening observed in this study is representative of pretensioned bridge girders in general.

REFERENCES

1. Powers, T. C. "The thermodynamics of volume change and creep," *Matériaux et Construction,* V.1, No.6, 1968, pp. 487-507.
2. Mindes, S., Young J., and Darwin D., "*Concrete*", Prentice Hall, 2003.
3. Neville, A. M., "*Properties of concrete*", 5^{th} Edition, Pearson Education, 2011.
4. American Association of State Highway and Transportation Officials (AASHTO). "*AASHTO LRFD Bridge Design Specification, Customary U.S. Units"*, 6th Edition. Washington, DC., 2012.

5. Garber, D., J. Gallardo, D. Deschenes, D. Dunkman, and O. Bayrak. "*Effect of New Prestress Loss Estimates on Pretensioned Concrete Bridge Girder Design*". Austin, Tx.:Center for Transportation Research, The University of Texas at Austin, 2012.
6. Gilbert, Raymond Ian, and Gianluca Ranzi. "*Time-dependent behaviour of concrete structures"*. Taylor & Francis, 2011.
7. ACI Committee 209R. "*Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*". American Concrete Institute, 1992 (Reapproved 2008).
8. Tadros, M. K., Al-Omaishi, N., Seguirant, S. J., and Gallt, J. G.. "*Prestress Losses in High-Strength Concrete Bridge Girders"*. Washington, D.C.: National Cooperative Highway Research Program., 2003.