# Evaluation of prestress losses in high-strength concrete bulb-tee girders for the Rigolets Pass Bridge

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## Editor's quick points

- One span of the Rigolets Pass Bridge containing four 131-ft-long (40 m), high-strength concrete bulb-tee girders was instrumented and monitored to obtain measured strain and temperature data.
- Measured girder prestress losses derived from concrete strains corrected for temperature and load effects were less than corresponding values calculated using *AASHTO LRFD Bridge Design Specifications*.
- Prestress losses calculated using both the approximate and refined estimates of time-dependent losses were evaluated relative to the measured losses.

The Louisiana Department of Transportation and Development (LADOTD) has been gradually introducing high-strength concrete into its bridge construction programs. At the same time, the Louisiana Transportation Research Center (LTRC) has been sponsoring research work to address design and construction issues related to the use of high-strength concrete.[1,2,3] Findings from these research endeavors have provided the LADOTD with valuable information related to material properties and the structural behavior of bridge components fabricated using high-strength concrete. This information has also provided the necessary technical basis for the continued implementation of high-strength concrete under a range of different bridge design conditions.

The construction of the Rigolets Pass Bridge located on US Route 90 and LA Route 433 in Orleans and St. Tammany Parishes, La., was recently completed. The Rigolets Pass Bridge (**Fig. 1**) is a 62-span bridge with a total length of 5489 ft (1673 m). High-strength concrete was used to produce the girders for two of the 62 bridge spans (spans 42 and 43). In conjunction with the Rigolets Pass Bridge project, a research program was conducted with the objective of monitoring the structural behavior of one of the two spans incorporating high-strength concrete.[4] Material-property studies were also included in the research program.

**Figure 1.** This photograph of Rigolets Pass Bridge was taken from the east end.

This paper presents results from this research, focusing on measured prestress losses and corresponding calculated values determined using the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[5]

## Girder fabrication

Spans 42 and 43 of the Rigolets Pass Bridge compose a continuous unit, with span lengths of 131.2 ft (40 m). These spans each incorporate four 78-in.-deep (1.98 m) bulb-tee (BT-78) girders at 12 ft 7 in. (3.83 m) spacing. The specified concrete compressive strengths for the girders were 6670 psi (46 MPa) at release and 10,000 psi (69 MPa) at 56 days. Both spans incorporate an 8-in.-thick (203 mm), cast-in-place, high-performance concrete (HPC) deck slab with a specified 28-day concrete compressive strength of 4200 psi (29 MPa). Span 43 of the Rigolets Pass Bridge was instrumented to measure concrete strains and temperatures at midspan. The four high-strength concrete girders used in span 43 were assigned specific alphanumeric designations (43A, 43B, 43C, and 43D) according to their position. The total width of the bridge deck is 45 ft 11 in. (14 m). **Figure 2** indicates typical details for the BT-78 girders.

All eight of the high-strength concrete BT-78 girders for spans 42 and 43 incorporate fifty-six 0.6-in.-diameter (15.2 mm), Grade 270 (270 ksi [1860 MPa]), low-relaxation, seven-wire strands conforming to ASTM A416.[6] Twelve of the 56 strands are debonded for lengths ranging from 6.6 ft to 39.4 ft (2 m to 12 m) at each end of the girders. The specified initial tensile force for each strand was 43.95 kip (195.5 kN), which corresponds to 75% of the specified minimum strand breaking strength.

The eight girders required for spans 42 and 43 were fabricated three at a time, using three separate castings or placements. **Table 1** shows the concrete mixture proportions used for the girders. Prior to placement, the measured slump of the concrete produced for all three castings ranged from 6½ in. to 9 in. (165 mm to 230 mm). Each girder required about 37.2 yd$^3$ (28.5 m$^3$) of concrete.

Immediately after placing the concrete, the bed was covered with a tarpaulin, which remained in place until the required release strength was achieved. None of the girders required steam curing. During the initial curing period, the concrete temperatures were monitored at the middle and both ends of the casting bed. **Figure 3** shows the concrete

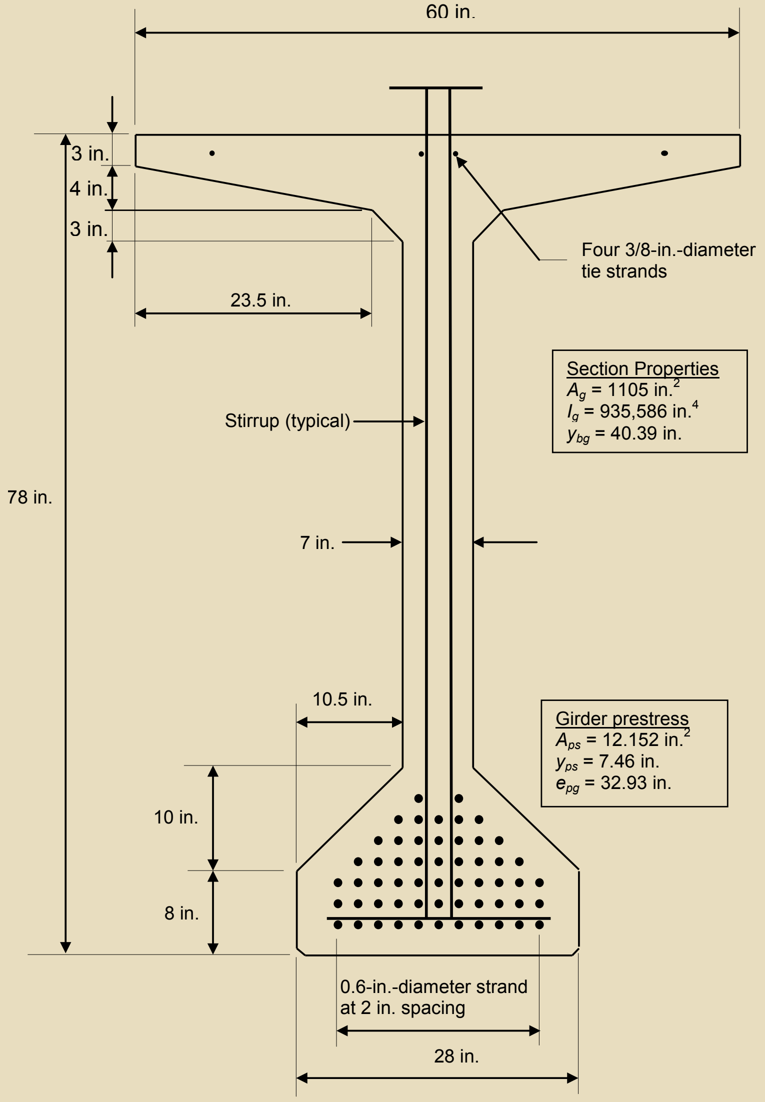


**Figure 2.** This drawing shows details of a typical BT-78 girder. Note: $A_g$ = gross cross-sectional area of girder section; $A_{ps}$ = area of prestressing steel; $e_{pg}$ = eccentricity of strands with respect to centroid of gross girder section; $I_g$ = moment of inertia of girder cross section; $y_{bg}$ = vertical distance between bottom of girder section and centroid of gross girder section; $y_{ps}$ = vertical distance between bottom of girder section and centroid of prestressing steel. 1 in. = 25.4 mm.

**Table 1.** Concrete mixture proportions for girders

| Material | Quantities/yd$^3$ |
|---|---|
| Portland cement Type III | 846 lb |
| Silica fume | 100 lb |
| Fine aggregate | 1149 lb |
| Coarse aggregate: limestone | 1866 lb |
| Water | 204 lb |
| Water-reducing admixture, ASTM C494, Type D | 38 fl oz |
| HRWRA, ASTM C494, Type F | 51 fl oz |
| Air entrainment | None |
| Water–cementitious materials ratio | 0.22 |

Note: HRWRA = high-range water-reducing admixture. 1 lb/yd$^3$ = 0.593 kg/m$^3$; 1 fl oz/yd$^3$ = 38.7 mL/m$^3$.

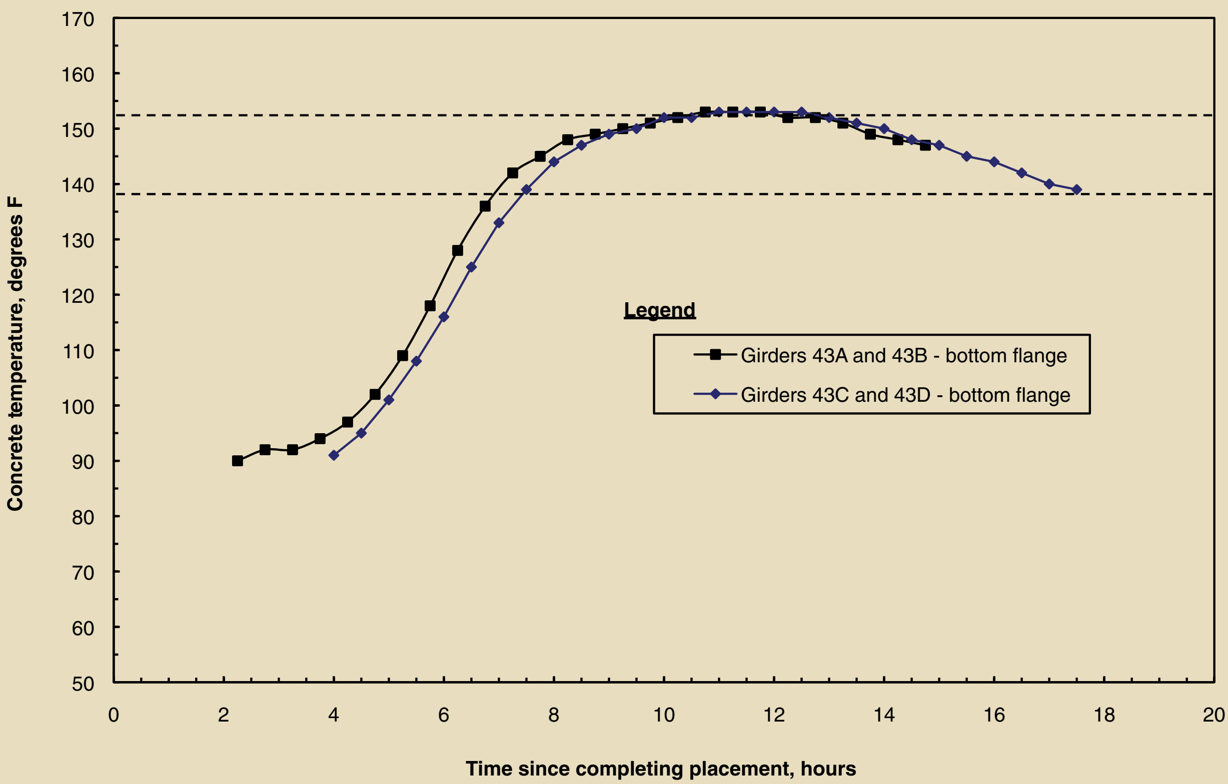


**Figure 3.** This figure depicts girder concrete curing temperatures measured from the time of placement until strand release. Note: °C = 5/9(°F – 32).

temperatures measured in the bottom flange of girders produced for span 43 during the initial curing period. The concrete in the bottom flange near midspan achieved a maximum temperature of about 153 °F (67 °C) for both castings. The maximum temperature was achieved about 12 hours after completion of concrete placement and then started to decrease at a rate of about 3 °F (1.5 °C) per hour.

The required release strength of 6670 psi (46 MPa) was typically achieved within 15 hours of completion of the casting. During the girder production runs for span 43, load cells were installed on 6 of the 56 prestressing strands between the dead-end bulkhead and anchorage chuck. These load cells were used to measure force levels at various time- or event-based intervals beginning at the time of initial tensioning and continuing until release. After stressing all strands, the measured force levels were within 5% of the specified 43.95 kip (195.5 kN) initial force, as required by the tolerance cited in the *PCI Manual for Quality Control*.[7] Prior to releasing the prestress, the measured force level in the strands ranged from 39.53 kip to 42.02 kip (175.8 kN to 186.9 kN) and averaged 40.49 kip (180.1 kN). This average is about 8% less than the specified initial force.

## Instrumentation

The high-strength concrete girders produced for span 43 of the Rigolets Pass Bridge were instrumented to obtain measured strain and temperature data. Five vibrating-wire strain gauges (VWSGs) were installed at the midspan of each of the four girders. Each VWSG incorporated a thermistor for measuring the concrete temperature associated with each strain measurement. Three of the VWSGs were installed in the lower flange at the elevation of the strand-group centroid. The remaining two gauges were installed at the center of gravity of the top flange, located about 3 in. (76 mm) down from the top surface of the girder. At the bridge site, seven additional VWSGs were installed in the HPC deck slab of span 43 at midspan. The VWSGs in the deck slab were installed directly above each girder and at the midpoint between each of the four girders. All seven of these VWSGs were installed at mid-depth of the 8-in.-thick (203 mm) deck slab. **Figure 4** shows the locations of all VWSG instrumentation installed in span 43 of the Rigolets Pass Bridge.

During construction, VWSG readings were made prior to strand release, after strand release, after girder storage, at a concrete age of 28 days, once a month for the next two months, before shipping, after erection, and before and after casting the deck-slab concrete. After casting the deck slab, all VWSGs installed in the girders and deck slab were read once a week for the first month and once a month until the on-site data acquisition system (DAS) was installed.

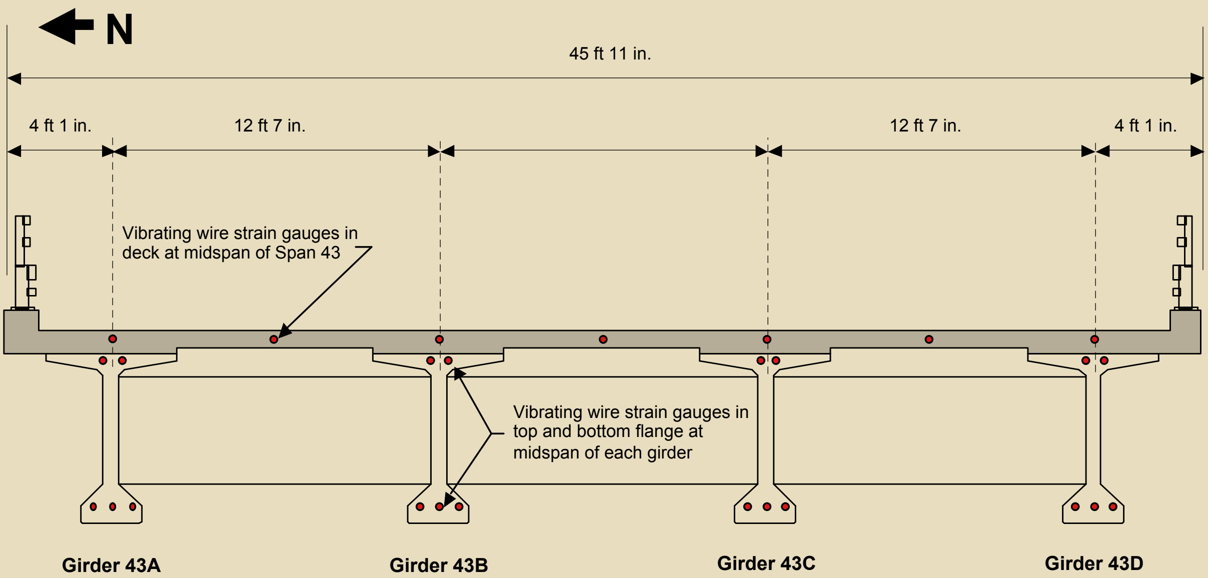


**Figure 4.** This figure shows a cross-sectional view of span 43 and vibrating-wire strain gauge instrumentation installed at midspan. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

**Figure 5.** This automated data acquisition system was installed on the south guardrail of span 43.

After completion of construction, the VWSGs were connected to an automated on-site DAS with remote-access capabilities. **Figure 5** shows a photograph of the DAS. Instrumentation for measuring ambient temperature and relative humidity conditions at the bridge site was also connected to the DAS. The DAS was programmed to record measured data for a monitoring period of one year. During this period, data from all instrumentation were read and stored once per hour. Stored data were downloaded via remote telephone modem access once a day and checked against previously stored data to ensure that the system was functioning properly. In addition, the measured data were posted to an interactive website created for the purpose of allowing the LADOTD to view and plot all the temperature and strain data recorded to date.

## Concrete material properties

Material property tests were performed on specimens representing concrete placed in the midspan region of each girder fabricated for span 43. Girder-concrete material property studies included unit weight, compressive strength (ASTM C39),[8] modulus of elasticity (ASTM C469),[9] and creep and shrinkage (ASTM C512).[10] Concrete cylinders used for unit weight, compressive strength, and modulus of elasticity were match cured to match the temperature in the lower flange of each corresponding girder. Other field-cured specimens were used for creep and shrinkage tests. Field-cured specimens were covered with plastic and stored adjacent to the casting bed. Just prior to release of the strands, all of

Table 2. Average girder concrete properties

| Concrete age, days | Average unit weight, lb/ft$^3$ | Average compressive strength, psi | Average modulus of elasticity, ksi |
|---|---|---|---|
| Release | 146.6 | 9250 | 6000 |
| 28 days | 146.2 | 10,850 | 6100 |
| 56 days | 146.0 | 11,260 | 6300 |
| 90 days | 146.6 | 11,670 | 6500 |

Note: 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa; 1 lb/ft$^3$ = 16.02 kg/m$^3$.

the match- and field-cured specimens were stripped from the molds.

**Table 2** provides the average measured unit weight, concrete compressive strength, and modulus of elasticity for the four girders of span 43. Each value represents the average of 12 individual tests (3 for each girder). The concrete used in each girder exhibited similar compressive strength at all test ages and achieved required release and 56-day strength levels cited in the project specifications for the Rigolets Pass Bridge.

Creep and shrinkage tests were performed on field-cured 6 in. × 12 in. (152 mm × 305 mm) cylinders representing concrete placed in the midspan region of girder 43D. The tests commenced at a concrete age of 3 days and were performed under ambient conditions of 73 ºF (23 ºC) and 50% relative humidity. The target applied load used for creep testing corresponded to 40% of the measured concrete compressive strength at the 3-day age of loading. Calculated midspan concrete stress at the centroid of the prestressing steel at the time of strand release $f_{cgp}$ for girder 43D also corresponded to about 40% of the measured concrete compressive strength. **Figures 6** and **7** show the measured creep coefficient, defined as the ratio of creep strain to initial strain, and shrinkage data, respectively.

Included in Fig. 6 and 7 are corresponding calculated values determined using provisions from the AASHTO LRFD specifications. According to article 5.4.2.3 of the AASHTO

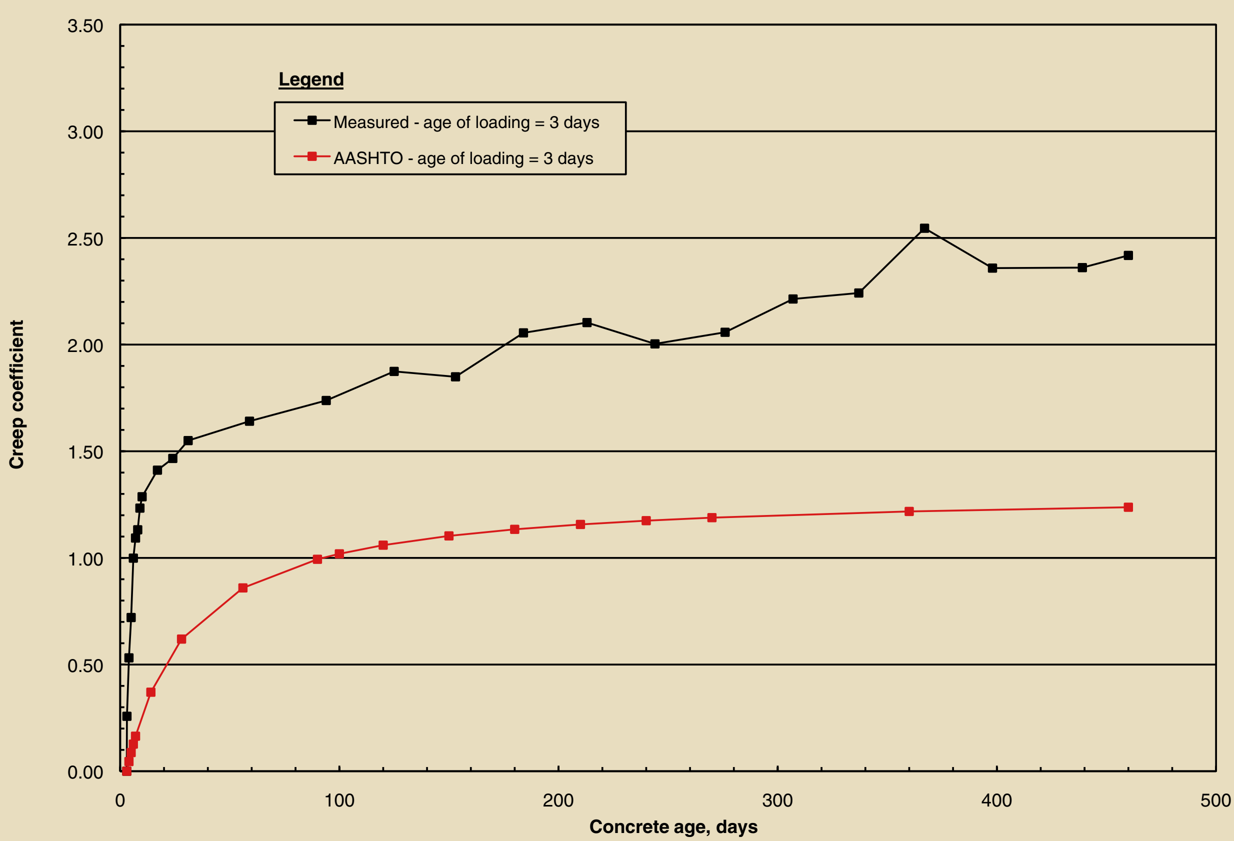


Figure 6. This figure shows plots of creep coefficient versus concrete age.

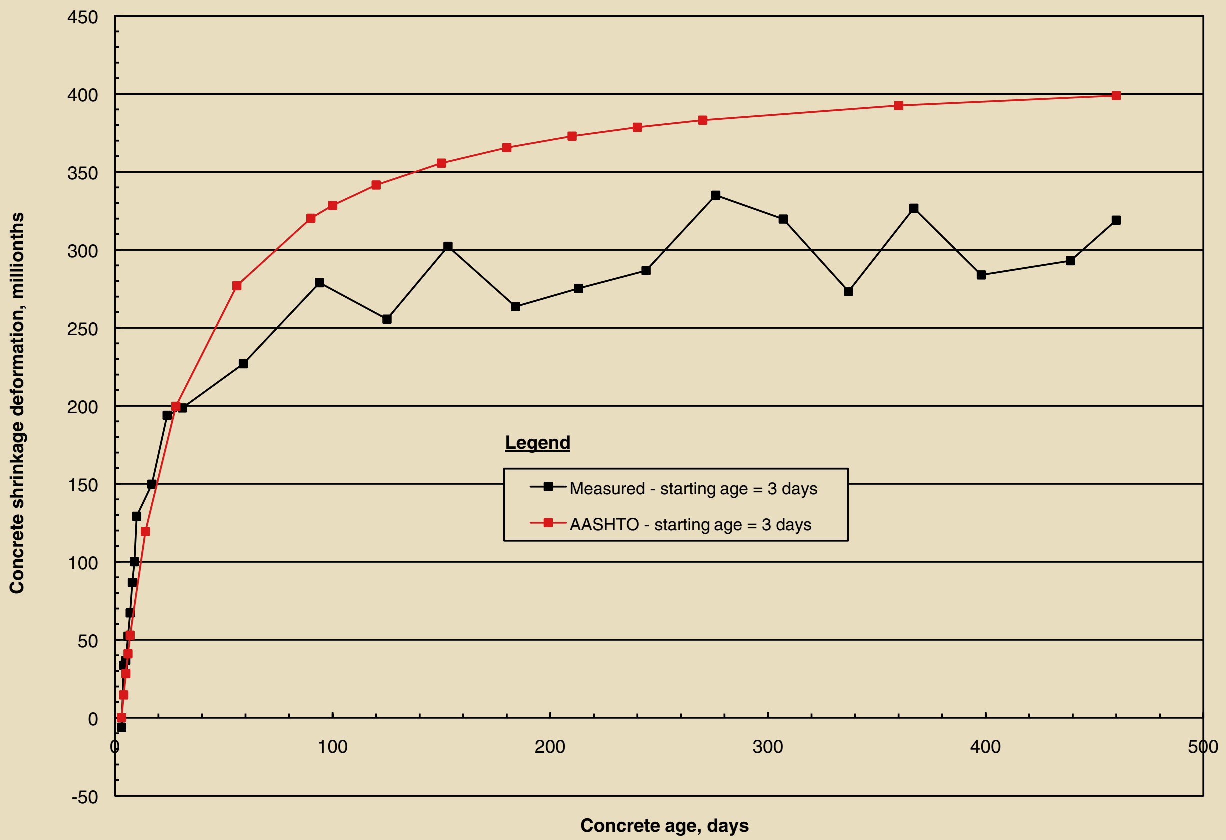


**Figure 7.** This figure shows plots of concrete shrinkage versus age.

LRFD specifications, when mixture-specific data are not available, estimates of creep and shrinkage may be made using the provisions of articles 5.4.2.3.2 and 5.4.2.3.3, respectively. Article 5.4.2.3.2 includes an equation for calculating the creep coefficient for various ages after initial loading. Article 5.4.2.3.3 includes an equation for calculating shrinkage at various concrete ages.

Based on the data (Fig. 6 and 7), the AASHTO LRFD specifications provisions for estimating creep and shrinkage (when mixture-specific data are not available) did not correlate well with the measured data. The final measured creep coefficient value for the 3-day age of loading was about 100% greater than the corresponding calculated AASHTO LRFD specifications estimated value. The final measured shrinkage value for the 3-day starting age was about 75% of the corresponding calculated AASHTO LRFD specifications estimated value. Consequently, for the field-cured cylinders made from the high-strength concrete batch placed in the midspan region of Girder 43D, the provisions of articles 5.4.2.3.2 and 5.4.2.3.3 of the AASHTO LRFD specifications underestimated the creep coefficient and overestimated shrinkage.

The field-cured cylinders used for the creep and shrinkage tests likely experienced different curing temperatures than the actual girder concrete temperatures (Fig. 3). Previous research has shown the importance of using match-cured test specimens to evaluate strength properties of precast concrete members made with high-strength concrete.[2] It is currently unknown how using match-cured cylinders for the creep and shrinkage tests would have affected the comparisons with AASHTO LRFD specifications estimated values. However, it is reasonable to assume that, by virtue

**Table 3.** Deck slab concrete properties

| Concrete age, days | Average unit weight, $lb/ft^3$ | Average compressive strength, psi | Average modulus of elasticity, ksi |
|---|---|---|---|
| 7 days | 144.2 | 5680 | 5150 |
| 28 days | 144.7 | 7760 | 5650 |
| 90 days | 144.6 | 8700 | 6450 |

Note: 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa; 1 $lb/ft^3$ = 16.02 $kg/m^3$.

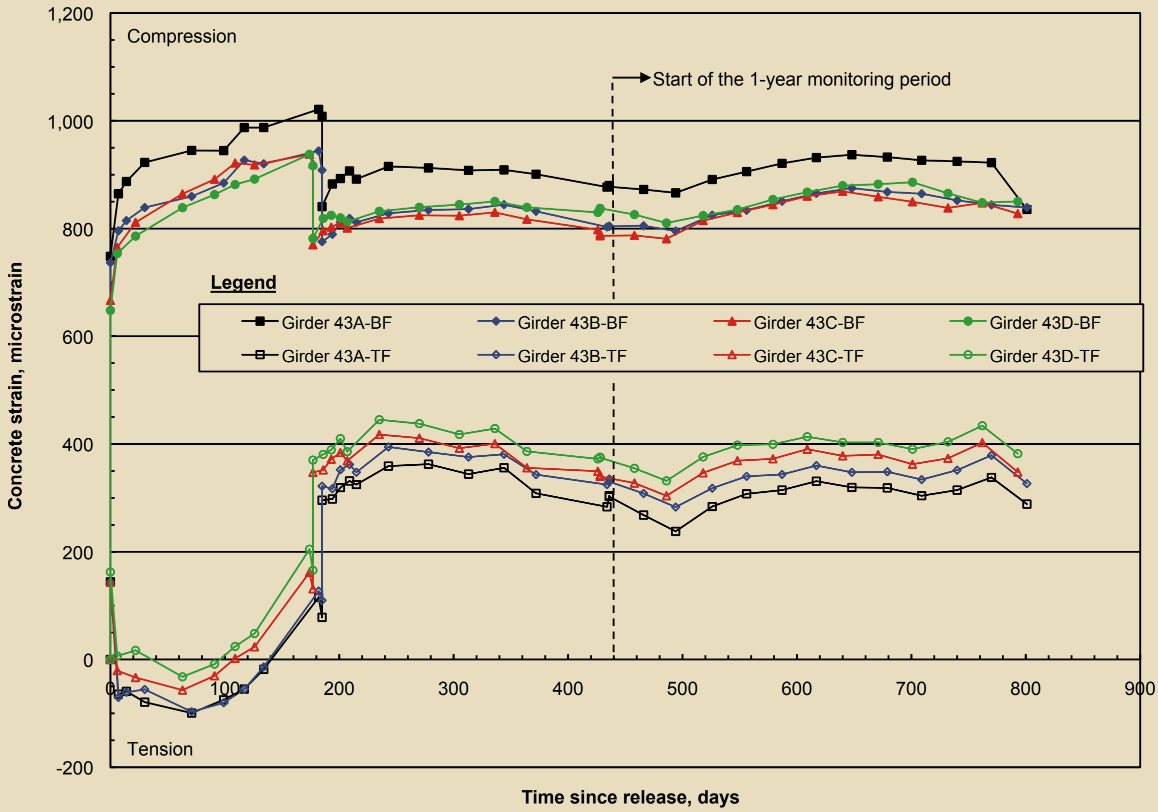


**Figure 8.** This figure shows plots of concrete strains measured in the top flange (TF) and bottom flange (BF) of each girder.

of their higher strength, match-cured cylinders would likely have produced lower creep and shrinkage deformations than the field-cured cylinders. The AASHTO LRFD specifications provisions for estimating creep and shrinkage (when mixture-specific data are not available) do not directly account for variations in initial curing conditions and the corresponding effect on concrete-strength development.

Material-property tests were performed on specimens representing HPC placed in the midspan region of the deck slab for span 43. Deck-slab concrete material property studies included unit weight, compressive strength (ASTM C39), and modulus of elasticity (ASTM C469). Concrete cylinder specimens used for material-property tests were field cured. Field-cured specimens were covered with plastic and stored adjacent to the bridge span for a period of seven days after concrete placement. **Table 3** provides average measured concrete properties for the deck slab of span 43. Each value reported in Table 3 represents the average of three individual tests. The concrete placed in the midspan region of span 43 achieved the required 28-day strength level cited in the project specifications for the Rigolets Pass Bridge.

## Measured concrete strains and prestress losses

VWSGs were installed at midspan in each of the four instrumented girders of span 43 (Fig. 4). The three VWSGs installed in the bottom flange at the center of gravity of the prestressing strands were used primarily to determine prestress losses. The two VWSGs installed in the top flange of each girder provided a second reference point for girder-section strain measurements, which was used as a means of validating the overall response to creep, shrinkage, temperature deformations, and external load effects. Each VWSG included a thermistor for measuring concrete temperature associated with each strain measurement. **Figure 8** shows plots of measured concrete strain versus time for VWSGs installed in the top and bottom flanges of each girder. Each plot represents the average of the data measured from all VWSGs installed in either the top or bottom flange of each girder.

The plotted average values (Fig. 8) were based on individual strain readings that were adjusted to a constant temperature. The temperature correction applied to the individual strain readings was calculated based on the measured concrete temperature associated with the reading, the average measured coefficient of thermal expansion of the

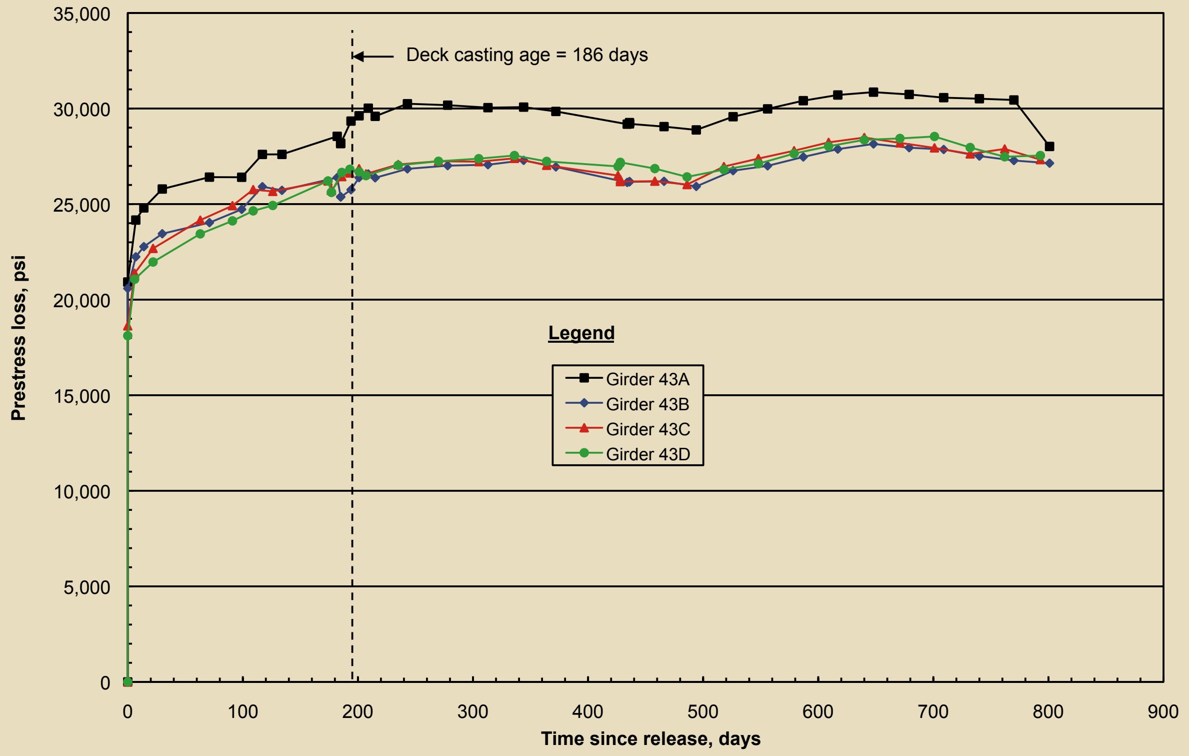


**Figure 9.** This figure shows plots of measured prestress losses derived from measured concrete strains in each girder. Note: 1 psi = 6.895 kPa.

girder concrete, and the coefficient of thermal expansion of the strain-gauge wire provided by the manufacturer. Therefore, the plotted average values represent corrected strains that theoretically do not include effects due to temperature change.

Strain readings taken just prior to release of the strands served as the zero reference for the concrete strains (Fig 8). After strand release, the compressive strains measured in the bottom flange (BF) gradually increased due to the effects of creep and shrinkage until the time when the deck slab was added. The corresponding measured strains in the top flange (TF) essentially mirrored the behavior of the bottom flange until about 80 days, when the readings exhibited maximum tension.

During the time period bounded by the dates of girder erection (between 126 and 134 days after strand release) and deck casting, the compressive strain in the bottom flange continued to increase while the top flange strains transitioned from slight tension to compression. The gradual load additions resulting from the diaphragms, formwork, and deck-slab reinforcing steel did not produce noticeable changes in the bottom-flange strains. However, the effects of these additions were evident in the measured top-flange strains. The data plotted in Fig. 8 indicate a marked reduction in the bottom-flange compressive strain and corresponding increase in the top-flange compressive strain occurring 185 days after release, denoting the elastic response due to placement of the deck-slab concrete.

The measured concrete strain changes at the center of gravity of the prestressing strands were used to quantify prestress loss. Using the compatibility assumption that the changes in concrete strain measured at the center of gravity of the prestressing strands are equal to the corresponding average steel strain changes, the measured values (Fig. 8) can be used to calculate prestress losses using the modulus of elasticity of the strand. When the bottom-flange strain data (Fig. 8) are edited to remove elastic-strain effects resulting from external loading and multiplied by the average measured modulus of elasticity of the prestressing strand equal to 27,950 ksi (193 GPa), the resulting plot of prestress loss in **Fig. 9** results. The data plotted in Fig. 9 represent the total measured prestress loss in each girder due to the combined effects of elastic shortening and time-dependent effects (concrete creep and shrinkage) for a time interval beginning at release of the prestressing strands and ending one year after construction was completed.

The measured prestress loss in Fig. 9 does not include any loss due to relaxation of the prestressing strands occurring after release because such losses occur without a corresponding change in strain. Relaxation losses occurring prior to release were also not considered, as is consistent with article 5.9.5.4.2c of the AASHTO LRFD specifications. However, based on the measured reduction in

**Table 4.** Prestress loss calculation scenarios

| Calculation parameter | Calculation scenario | |
|---|---|---|
| | Design | Constructed |
| Girder concrete $f'_{ci}$, psi | 6670 | 9250 |
| Girder concrete $E_{ci}$, ksi | 4951* | 6000 |
| Girder concrete $f'_c$, psi | 10,000 | 11,260 (56 days) |
| Girder concrete $E_c$, ksi | 6062* | 6300 (56 days) |
| Girder unit weight, lb/ft$^3$ | 155 | 153 (56 days)† |
| Deck slab $f'_c$, psi | 4200 | 7760 (28 days) |
| Deck slab $E_c$, ksi | 3929* | 5650 (28 days) |
| Deck slab unit weight, lb/ft$^3$ | 150 | 153 (28 days)‡ |
| Haunch thickness, in. | 2 | 2 |
| Strand stress prior to transfer $f_{pbt}$, ksi | 202.50§ | 186.59** |
| Strand modulus of elasticity, ksi | 28,000 | 27,950 |

*Calculated based on an assumed concrete unit weight of 150 lb/ft$^3$ and $E_c = w_c^{1.5}\, 33\sqrt{f'_c}$

†Based on average measured concrete unit weight at 56 days equal to 146 lb/ft$^3$ plus an estimated additional 7 lb/ft$^3$ to account for the weight of the prestressing concrete strand and reinforcement

‡Based on average measured concrete unit weight at 28 days equal to 145 lb/ft$^3$ plus an estimated additional 8 lb/ft$^3$ to account for the weight of the deck-slab reinforcement

§Assumed equal to 0.75 times specified tensile strength of prestressing steel $f_{pu}$

**Based on average measured strand load prior to release equal to 40.39 kip

Note: $E_c$ = modulus elasticity of girder concrete; $E_{ci}$ = modulus elasticity of girder concrete at prestress transfer; $f'_c$ = specified compressive strength of concrete used in design; $f'_{ci}$ = specified compressive strength of concrete at prestress transfer; $f_{pbt}$ = stress in prestressing steel immediately prior to transfer; $w_c$ = unit weight of concrete. 1 in. = 25.4 mm; 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa; 1 lb/ft$^3$ = 16.02 kg/m$^3$.

**Table 5.** Measured and calculated prestress losses using *AASHTO LRFD Bridge Design Specifications*

| Prestress loss component | Measured losses, psi | Calculated losses per articles 5.9.5.2 and 5.9.5.3 (approximate method), psi | | Calculated losses per articles 5.9.5.2 and 5.9.5.4 (refined method), psi | |
|---|---|---|---|---|---|
| | | Design | Constructed | Design | Constructed |
| $\Delta fp_{ES}$ | 19,560 | 19,840 | 15,100 | 19,840 | 15,100 |
| $\Delta fp_{LT}$ | 8710* | 21,160† | 15,140† | 20,150† | 12,790† |
| Total | 28,270 | 41,000 | 30,240 | 39,990 | 27,890 |

*Based on maximum total prestress loss measured about 650 days after strand release

†For comparison with the measured data, relaxation losses $\Delta f_{pR1}$ and $\Delta f_{pR2}$ are omitted.

Note: $\Delta f_{pES}$ = loss in prestressing steel stress due to elastic shortening; $\Delta f_{pLT}$ = long-term prestress loss due to creep of concrete, shrinkage of concrete, and relaxation of steel. 1 psi = 6.895 kPa.

prestress force occurring from the time of initial tensioning until strand release, it is apparent that these prerelease reductions constitute a source of prestress loss not presently addressed in the AASHTO LRFD specifications.

The measured prestress loss for each girder gradually increased from the time of strand release until about 340 days after release (Fig. 9). From that point, the measured losses decreased slightly over the next 160 days and then began to increase again from about 500 days until reaching an apparent maximum value about 650 days after release. There are two explanations for the observed fluctuation in measured prestress loss. One explanation is that the temperature compensation applied to the measured strains does not completely eliminate temperature effects, causing seasonal temperature variations to have a slight, yet noticeable, effect on the measured prestress loss. The second explanation relates to shrinkage of the deck-slab concrete. Deck-slab shrinkage will tend to reduce bottom-flange compressive strains. Consequently, the measured prestress losses plotted in Fig. 9 beyond 185 days include increases due to the combined effect of continuing creep and shrinkage of the girder concrete and reductions due to the progression of deck-slab shrinkage. Although the measured prestress losses likely include some negating effects due to deck-slab shrinkage, the observed fluctuations appear to be more consistent with the seasonal temperature variation causation scenario than the deck-slab shrinkage causation scenario.

## Calculated prestress losses

For comparison with the measured data, prestress losses were calculated for a typical interior girder using the provisions of the AASHTO LRFD specifications (including 2008 interim revisions[11]) defined in articles 5.9.5.2.3, 5.9.5.3, and 5.9.5.4. Prestress-loss calculations were made to evaluate two different scenarios. The first scenario represents the design condition, where the calculated loss parameters are based on specified material properties. The second scenario represents the constructed condition, where the calculated loss parameters are based on measured material properties. **Table 4** gives specific conditions associated with the two prestress-loss calculation scenarios.

**Table 5** gives the measured and calculated prestress losses for the interior girders of span 43. The tabulated prestress losses are presented in terms of two loss components: elastic shortening $\Delta f_{pES}$ and time-dependent losses $\Delta f_{pLT}$. Measured elastic shortening prestress loss was determined based on the average initial concrete strain change resulting from release of the prestress force. The total measured prestress loss value (Table 5) is based on the average maximum prestress loss, typically occurring about 650 days after release. Measured prestress loss due to the combined effects of girder concrete shrinkage and creep was assumed equal to the total loss less the elastic shortening loss.

Calculated elastic shortening prestress loss $\Delta f_{pES}$ was evaluated using the provisions given in article 5.9.5.2 of the AASHTO LRFD specifications. Calculated time-dependent prestress losses $\Delta f_{pLT}$ were evaluated using the provisions given in article 5.9.5.3 (approximate method) and article 5.9.5.4 (refined method) of the AASHTO LRFD specifications. The appendix shows prestress-loss calculations for the design scenario.

### Elastic shortening loss

The elastic shortening prestress loss component $\Delta f_{pES}$ for both design and constructed conditions was calculated using Eq. (C5.9.5.2.3a-1) of the AASHTO LRFD specifications commentary to avoid having to initially estimate the prestress force after transfer and perform multiple iterations with Eq. (5.9.5.2.3a-1). In Eq. (C5.9.5.2.3a-1), the stress in the strand immediately prior to transfer $f_{pbt}$ was taken as 202,500 psi (1396 MPa) for the design condition and 186,590 psi (1287 MPa) for the constructed condition. The design $f_{pbt}$ value corresponds to 75% of the specified prestressing strand tensile strength. The constructed $f_{pbt}$ value corresponds to the average measured strand force prior to release of 40.49 kip (180.1 kN) divided by the nominal strand area. The resulting $\Delta f_{pES}$ values were subsequently confirmed using Eq. (5.9.5.2.3a-1).

The measured elastic shortening prestress loss correlated well with the corresponding design calculated loss (Table 5). Correlation between the measured elastic shortening loss and corresponding constructed calculated loss was not as favorable. The calculated elastic shortening loss is dependent on the concrete modulus of elasticity at the time of strand release. The design elastic shortening loss was calculated using an assumed concrete modulus-of-elasticity value of 4950 ksi (34 GPa), while the constructed elastic shortening value was calculated using a measured concrete modulus-of-elasticity value of 6000 ksi (41 GPa) (Table 4). The modulus-of-elasticity value used in the constructed elastic shortening loss calculation was measured several hours after the time that release actually took place and, therefore, may have been somewhat greater than the actual concrete modulus of elasticity at the time of strand release.

Another possible explanation for the discrepancy between the measured and constructed elastic shortening prestress loss is that the measured concrete strains resulting from release included some creep and shrinkage loss. The time duration between the pre- and postrelease strain readings for the four instrumented girders was typically about 1.5 hr. The postrelease readings were typically taken within 20 min of the conclusion of strand release. While efforts were made to minimize the elapsed time between the pre- and postrelease strain measurements, it is likely that the resulting measured change in strain occurring over this time interval included some creep and shrinkage loss.

### Time-dependent loss

Time-dependent prestress losses $\Delta f_{pLT}$ were calculated using both the approximate method defined in article 5.9.5.3 and the refined method defined in article 5.9.5.4 of the AASHTO LRFD specifications. Table 5 shows the calculated $\Delta f_{pLT}$ losses using both analysis methods for the design and constructed conditions. For both analysis methods, the girder concrete age at deck casting was taken as 186 days. For the design condition, the approximate and refined methods produced comparable $\Delta f_{pLT}$ losses. Therefore, for the conditions associated with span 43 of the Rigolets Pass Bridge, use of the more rigorous and

**Table 6.** Calculated time-dependent prestress loss components using article 5.9.5.4 of the *AASHTO LRFD Bridge Design Specifications*

| | Time-dependent prestress loss component | Calculated time-dependent loss per article 5.9.5.4, psi | |
|---|---|---|---|
| | | Design | Constructed |
| Time of transfer to time of deck placement | $\Delta f_{pSR}$ | 5510 | 4670 |
| | $\Delta f_{pCR}$ | 15,670 | 10,140 |
| | $\Delta f_{pR1}$ | 0 | 0 |
| Time of deck placement to final time | $\Delta f_{pSD}$ | 1050 | 570 |
| | $\Delta f_{pCD}$ | -980 | -1660 |
| | $\Delta f_{pR2}$ | 0 | 0 |
| | $\Delta f_{pSS}$ | -1100 | -930 |
| | Total $\Delta f_{pLT}$ | 20,150 | 12,790 |

Note: $\Delta f_{pCD}$ = prestress loss due to creep of girder concrete between time of deck placement and final time; $\Delta f_{pCR}$ = prestress loss due to creep of girder concrete between transfer and deck placement; $\Delta f_{pLT}$ = long-term prestress loss due to creep of concrete, shrinkage of concrete, and relaxation of steel; $\Delta f_{pR1}$ = prestress loss due to relaxation of prestressing strands between transfer and deck placement; $\Delta f_{pR2}$ = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time; $\Delta f_{pSD}$ = prestress loss due to shrinkage of girder concrete between time of deck placement and final time; $\Delta f_{pSR}$ = prestress loss due to shrinkage of girder concrete between transfer and deck placement; $\Delta f_{pSS}$ = prestress loss due to shrinkage of deck composite section. 1 psi = 6.895 kPa.

time-consuming refined analysis method for calculation of the $\Delta f_{pLT}$ loss did not appear warranted. However, for the constructed condition, the refined analysis method produced a calculated $\Delta f_{pLT}$ loss that was about 16% less than the approximate analysis method.

Both the approximate and refined provisions resulted in calculated $\Delta f_{pLT}$ losses that were about 1½ to 2½ times greater than the average creep and shrinkage loss measured in the interior girders of span 43 since the time of release (Table 5). The calculated prestress losses should be viewed as ultimate (end of service life) values. Therefore, total losses measured within the first few years of service would be expected to be somewhat less than calculated values. However, based on the measured prestress loss plots (Fig. 9), it appears that the vast majority of the ultimate loss has already occurred.

**Table 6** shows a breakdown of the calculated time-dependent loss components derived using the refined method defined in article 5.9.5.4 of the AASHTO LRFD specifications for the design and constructed conditions. The refined provisions for calculating $\Delta f_{pLT}$ account for prestress gain due to shrinkage of the composite deck slab $\Delta f_{pSS}$ by subtracting this effect from the other long-term loss parameters. Conversely, the measured prestress-loss data (Fig. 9 and Table 5) likely include the effects of deck-slab shrinkage. Recognizing that the deck slab was added 185 days after strand release (concrete age of 186 days) and that the compressive strain measured in the bottom flange of the girders continued to increase at a uniform rate both before and beyond 185 days, the prestress gain resulting from deck-slab shrinkage does not appear to have had a significant effect on measured girder prestress losses. This finding is supported by the observation that the calculated $\Delta f_{pSS}$ term (Table 6) for both the design and constructed conditions is only about 1000 psi (6.9 MPa).

Due to the elapsed time between the pre- and postrelease strain measurements, a small portion of the measured elastic shortening loss may have included some creep and shrinkage effects. It is also possible that other conditions, such as precision of the applied temperature corrections to strain readings, may have had a slight effect. However, it does not appear likely that the noted discrepancy between measured and calculated time-dependent losses can be attributed to these circumstances.

Based on the results from the creep and shrinkage tests performed on representative samples of the girder concrete, it appears that the discrepancy between measured and calculated time-dependent losses is most likely due to inaccuracy of the various factors and coefficients used in both the approximate and refined analysis methods. Results from creep and shrinkage tests performed on field-cured cylinders indicated that the provisions of AASHTO LRFD specifications articles 5.4.2.3.2 and 5.4.2.3.3 underestimated creep coefficient and overestimated shrinkage. However, the calculated $\Delta f_{pLT}$ loss using both approximate and refined methods overestimated the combined effects of creep and shrinkage loss by a significant margin. The apparent lack of correlation between cylinder test results and comparisons between measured and calculated $\Delta f_{pLT}$ loss suggests that the models relating creep coefficient and shrinkage and corresponding prestress-loss components do not adapt well to the conditions associated with span 43 of the Rigolets Pass Bridge.

## Discussion of current AASHTO LRFD specifications prestress-loss provisions and corresponding recommendations

The AASHTO LRFD specifications provisions for estimating prestress losses were refined based largely on research described in National Cooperative Highway Research Program (NCHRP) report 496, "Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders."[12] The current AASHTO LRFD specifications methodology was used by the authors as a basis of comparison with measured prestress losses in span 43 of the Rigolets Pass Bridge. Comparisons between measured and calculated prestress losses were not favorable, with the calculated losses based on the refined method for the design condition being about 40% greater than the measured loss. For this comparison, the greatest source of error was related to estimation of the time-dependent losses. Because the measured loss includes the combined effects of both creep and shrinkage, it is not possible to discern which of the loss parameters produced the greatest discrepancy with corresponding calculated values.

The provisions in article 5.9.5.4 for refined estimates of time-dependent losses incorporate numerous terms and equations used primarily to define shrinkage, creep, and relaxation losses. In the current form, interpretation and use of these provisions is difficult, and the existing commentary offers little clarification. Issues with sign conventions and equation terms that have not been clearly defined make these provisions extremely difficult to use without further guidance. Useful guidance related to implementation of the provisions in article 5.9.5.4 can be found in a paper by Al-Omaishi, Tadros, and Seguirant.[13] However, changes to the AASHTO LRFD specifications provisions for estimating prestress losses are still needed so that users will not have to look to other sources for interpretation.

The following comments relate to the current AASHTO LRFD specifications and will hopefully be useful in future refinement efforts:

- Article 5.9.5.2.3 contains an equation for elastic shortening prestress loss that has essentially remained unchanged from what has appeared for decades in earlier editions of AASHTO LRFD specifications. However, the commentary that now accompanies this equation states that an iterative approach should be used to ensure that the initial assumed strand stress used to calculate $f_{cgp}$ is consistent with the resulting calculated $\Delta f_{pES}$ value. Use of commentary Eq. (C5.9.5.2.3a-1) is offered as an alternative to the iterative approach. Clearly, the iterative approach to calculate $\Delta f_{pES}$ is more technically correct than the previous methodology where $f_{cgp}$ was calculated based on one assumed strand stress value. However, it is questionable whether the additional iterative calculation effort is warranted. In an effort to simplify, the authors recommend that the commentary equation be used instead of Eq. (5.9.5.2.3a-1).

- The fourth paragraph of the commentary that currently accompanies article 5.9.5.2.3 contains discussion related to elastic gains and the use of transformed section properties. This paragraph has the potential to cause confusion. Because the accompanying equations do not require the use of transformed section properties and the weight of the girder is the only elastic effect considered in the calculation of $\Delta f_{pES}$, most of the commentary discussion appears to be inapplicable and unnecessary.

- The time-dependent-loss analysis method presented in article 5.9.5.3 uses a combination of multipliers and factors that were derived to approximate the terms used in the refined analysis method presented in article 5.9.5.4. The refined analysis method uses estimates of creep and shrinkage derived from the provisions of articles 5.4.2.3.2 and 5.4.2.3.3. Although findings from this study suggest that the current provisions yielded an overall conservative estimate (overestimation) of prestress loss, the discrepancy between calculated and measured time-dependent loss suggests that the recent refinements incorporated in article 5.9.5.4 are unnecessary and imply a degree of precision that does not occur in practice. If the use of article 5.9.5.4 provisions is to be continued, it is recommended that further refinements be made to account for a wider range of concrete mixture proportions and bridge design conditions.

- The provisions of articles 5.4.2.3.2 and 5.4.2.3.3 account for variations in relative humidity, volume-to-surface ratio, concrete strength, and age of loading. However, other factors known to have an effect on creep and/or shrinkage properties are either not considered or are addressed indirectly. These include type and quantity of cement replacement materials, initial curing conditions, initial concrete stress level, and aggregate properties. Findings from this research indicated that the high-strength concrete girders of the Rigolets Pass Bridge (incorporating about 10% silica fume by weight of cementitious materials) had measured creep and shrinkage prestress losses that were one-third to one-half of the time-dependent loss calculated using the existing AASHTO LRFD specifications provisions. According to NCHRP report 496, research using the New Hampshire mixture proportions incorporating silica fume produced creep and shrinkage deformations that were considerably less than corresponding values calculated using the provisions of articles 5.4.2.3.2 and 5.4.2.3.3.[12] These two examples indicate that the current AASHTO LRFD specifications provisions will not always provide accurate estimates of time-dependent prestress losses. Although the current AASHTO LRFD specifications provisions may provide a better prediction of prestress losses than the earlier methodologies, the level of effort now required for implementation of provisions for refined estimates of time-dependent losses is not warranted and should be clarified and simplified. In addition, it appears that more work is needed if the provisions of articles 5.4.2.3.2 and 5.4.2.3.3 and the resulting calculated creep coefficients and shrinkage deformations are to more closely model measured test values over a wider range of design and/or material conditions.

- According to Eq. (5.9.5.4.1-1), the $\Delta f_{pSS}$ term used to define prestress gain resulting from shrinkage of the deck-slab concrete is subtracted from the shrinkage, creep, and steel relaxation loss terms. However, Eq. (5.9.5.4.3d-1) produces a negative $\Delta f_{pSS}$ value. Therefore, it appears that the sign preceding the $\Delta f_{pSS}$ term in Eq. (5.9.5.4.1-1) is incorrect.

- The definition of the $e_{pc}$ term presented in article 5.9.5.4.3 should be eccentricity of strands with respect to centroid of gross composite section. The words "gross composite concrete section" should be used as they are in the subsequent definitions for the $A_c$ and $I_c$ terms. Also, the words "at service" should be inserted at the end of the definition of the $A_c$ term to be consistent with the definition of $I_c$. $A_c$ is the area of section calculated using the gross composite concrete section properties of the girder and the deck (including the haunch) and the deck-to-girder modular ratio at service, and $I_c$ is the moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck (including the haunch) and the deck-to-girder modular ratio at service.

- The equation for prestress loss due to creep of girder concrete between time of deck placement and final time $\Delta f_{pCD}$ (Eq. [5.9.5.4.3b-1]) contains one error and one term that requires further clarification. The first $\Psi_b$ term should be moved inside the square brackets to read as $[\Psi_b(t_f, t_i) - \Psi_b(t_d, t_i)]$, where $\Psi_b(t_f, t_i)$ is the girder creep coefficient at final time due to loading introduced at transfer, and $\Psi_b(t_d, t_i)$ is the girder creep coefficient at time of deck placement due to loading introduced at transfer. Clarification is needed regarding the second part of the equation incorporating the $\Delta f_{cd}$ term. A definition has been provided for the $\Delta f_{cd}$ term: change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement combined with deck weight (acting on noncomposite section) and superimposed loads (acting on composite section). However, no further guidance or equations are currently included. An equation for $\Delta f_{cd}$ can be found in the paper by Al-Omaishi, Tadros, and Seguirant. This equation accounts for changes in concrete stress resulting from previous prestress loss expressed as a force acting on the noncomposite, nontransformed girder section; the deck slab bending moment acting on the girder section with transformed strand; and the superimposed dead-load moment acting on the composite concrete section with transformed strand. Each of these effects should combine under the same numerical sign, as all function to either reduce subsequent creep loss or result in an elastic gain. In the current form of the equation for $\Delta f_{cd}$ provided in Al-Omaishi, Tadros, and Seguirant, the elastic gains from the weight of the deck slab and superimposed dead load are calculated using noncomposite and composite section properties with transformed strand. This approach greatly increases the level of effort required to calculate $\Delta f_{pCD}$. Although an equation for $\Delta f_{cd}$ should be provided in the AASHTO provisions, an effort should be made to simplify so that the user need not perform multiple calculations to define both transformed and nontransformed section properties for both noncomposite and composite sections.

- Equation (5.9.5.4.3b-1) currently indicates a plus sign between the first and second parts of the equation. If this sign convention is to be used, then it is important to note that the sign associated with the $\Delta f_{cd}$ term should be negative.

- The current AASHTO LRFD specifications prestress loss provisions include allowances for prestress gain caused by shrinkage of the deck-slab concrete $\Delta f_{pSS}$. Prestress loss calculations performed as part of this study indicated that the $\Delta f_{pSS}$ term only accounted for a gain of about 1 ksi (7 MPa). In the numerical design example by Al-Omaishi, Tadros, and Seguirant, the calculated $\Delta f_{pSS}$ loss was 1.21 ksi (8.34 MPa). Consequently, consideration should be given to eliminating the $\Delta f_{pSS}$ term in an effort to further simplify the prestress-loss calculations.

## Acknowledgments

The research described in this paper was sponsored by the LTRC. The project team acknowledges the following people and organizations that contributed to the success of this research: Paul B. Fossier, LADOTD bridge engineer administrator, for technical guidance throughout the project; John Eggers, Sadi Torres, and Randy Young, LTRC concrete testing laboratory, for assisting with the material testing program and instrumentation readings; the staff of Gulf Coast Pre-Stress Inc. of Pass Christian, Miss., especially Don Theobald, for their patience during fabrication, storage, and shipping of the instrumented bridge girders; and the staff of Massman Construction Co. of Kansas City, Mo., especially Steve Hayes and Terrence Colombatto, for their patience and assistance during construction of the instrumented bridge span. The authors also thank the *PCI Journal* reviewers for their thoughtful and constructive comments.

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## Notation

$A_c$ = area of section calculated using the gross composite concrete section properties of the girder and the deck (including the haunch) and the deck-to-girder modular ratio at service

$A_g$ = gross cross-sectional area of girder section

$A_{ps}$ = area of prestressing steel

$e_{pc}$ = eccentricity of strands with respect to centroid of gross composite section

$e_{pg}$ = eccentricity of strands with respect to centroid of gross girder section

$E_c$ = modulus elasticity of girder concrete

$E_{ci}$ = modulus elasticity of girder concrete at prestress transfer

$f'_c$ = specified compressive strength of concrete used in design

$f_{cgp}$ = concrete stress at center of gravity of prestressing steel due to prestress force at transfer and self-weight of member at sections of maximum moment

$f'_{ci}$ = specified compressive strength of concrete at prestress transfer

$f_{pbt}$ = stress in prestressing steel immediately prior to transfer

$f_{pu}$ = specified tensile strength of prestressing steel

$I_c$ = moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck, including the haunch, and the deck-to-girder modular ratio at service

$I_g$ = moment of inertia of girder cross section

$w_c$ = unit weight of concrete

$\Delta f_{cd}$ = change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement, combined with deck weight (acting on noncomposite section) and superimposed loads (acting on composite section)

$\Delta f_{pCD}$ = prestress loss due to creep of girder concrete between time of deck placement and final time

$\Delta f_{pCR}$ = prestress loss due to creep of girder concrete between transfer and deck placement

$\Delta f_{pES}$ = loss in prestressing steel stress due to elastic shortening

$\Delta f_{pLT}$ = long-term prestress loss due to creep of concrete, shrinkage of concrete, and relaxation of steel

$\Delta f_{pR1}$ = prestress loss due to relaxation of prestressing strands between transfer and deck placement

$\Delta f_{pR2}$ = prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time

$\Delta f_{pSD}$ = prestress loss due to shrinkage of girder concrete between time of deck placement and final time

$\Delta f_{pSR}$ = prestress loss due to shrinkage of girder concrete between transfer and deck placement

$\Delta f_{pSS}$ = prestress loss due to shrinkage of deck composite section

$\Psi_b(t_d, t_i)$ = girder creep coefficient at time of deck placement due to loading introduced at transfer

$\Psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer

# Appendix: Prestress loss calculations—design condition

## Girder section properties

$A_g$ = 1105 in.$^2$ (712,902 mm$^2$)

$I_g$ = 935,586 in.$^4$ (3.894202 × 10$^{11}$ mm$^4$)

$y_{bg}$ = vertical distance between bottom of girder section and centroid of gross girder section = 40.39 in. (1026 mm)

$V/S$ = volume-to-surface ratio of girder or deck slab (including haunch) = 3.74 (includes top and bottom surfaces of girder)

## Girder material properties

$f_c'$ = 10,000 psi (69 MPa)

$E_c$ = 6062 ksi (41,796 MPa)

$f_{ci}'$ = 6670 psi (46 MPa)

$E_{ci}$ = 4951 ksi (34,136 MPa)

$w_g$ = unit weight of girder concrete = 155 lb/ft$^3$ (2483 kg/m$^3$)

$E_p$ = modulus elasticity of prestressing steel = 28,000 ksi (193,053 MPa)

$f_{pu}$ = 270 ksi (1862 MPa)

## Girder prestress

$A_{ps}$ = 56 × 0.217 = 12.152 in.$^2$ (7840 mm$^2$)

$y_{ps}$ = vertical distance between bottom of girder section and centroid of prestressing steel = 7.46 in. (190 mm) (strand pattern in Fig. 2)

$e_{pg}$ = 40.39 – 7.46 = 32.93 in. (836 mm)

$f_{pbt}$ = $0.75f_{pu}$ = 0.75 × 270 = 202.500 ksi (1396 MPa)

## Girder self-weight moment $M_g$

Girder span at release = 130.8 ft (39.87 m)

$$M_g = \frac{(1105/144)(155)(130.8)^2}{(8)(1000)} = 2543.65 \text{ kip-ft (3449 kN-m)}$$

## Transformed girder section properties

Transformed strand area = (12.152)[(28,000/6062) – 1] = 43.98 in.$^2$ (1117 mm$^2$)

$y_{btg}$ = vertical distance between bottom of girder section and centroid of transformed girder section = 44,959.04/1148.97 = 39.13 in. (994 mm)

**Table A1.** Transformed girder section properties at service

| Section | Area, in.$^2$ | $y_b$, in. | $Ay_b$, in.$^3$ | $I$, in.$^4$ | $d$, in. | $Ad^2$, in.$^4$ | $I + Ad^2$, in.$^4$ |
|---|---|---|---|---|---|---|---|
| Girder | 1105.00 | 40.39 | 44,630.95 | 935,586 | –1.26 | 1754.30 | 937,340 |
| Strand* | 43.98 | 7.46 | 328.09 | 0 | 31.67 | 44,111.45 | 44,111 |
| $\Sigma$ | 1148.97 | n.a. | 44,959.04 | n.a. | n.a. | n.a. | 981,451 |

*Strands are assumed to be grouped at one location and corresponding $I$ is neglected

Note: $A$ = cross-sectional area of component; $d$ = vertical distance from centroid of component area to centroid of composite section; $I$ = moment of inertia of component; n.a. = not applicable; $y_b$ = vertical distance between bottom of composite section and centroid of component; $\Sigma$.
1 in. = 25.4 mm; 1 in.$^2$ = 645 mm$^2$; 1 in.$^3$ = 16,387 mm$^3$; 1 in.$^4$ = 416,231 mm$^4$.

$e_{ptg}$ = eccentricity of strands with respect to centroid of transformed girder section

= 39.13 – 7.46 = 31.67 in. (804 mm)

## Deck-slab material properties

$f'_c$ = 4200 psi (29 MPa)

$E_{cd}$ = modulus elasticity of deck-slab concrete = 3929 ksi (27,090 MPa)

$w_d$ = unit weight of deck-slab concrete = 150 lb/ft$^3$ (2403 kg/m$^3$)

## Composite section properties

Girder spacing = 12.583 ft (3.84 m)

Slab thickness = 8 in. (203 mm)

Haunch thickness = 2 in. (51 mm)

Modular ratio = 3929/6062 = 0.65

Transformed deck area = (0.65)(12.583)(8)(12) = 785.20 in.$^2$ (506,580 mm$^2$)

Transformed haunch area = (0.65)(60)(2) = 78.00 in.$^2$ (50,322 mm$^2$)

**Table A2.** Gross composite section properties

| Section | Area, in.$^2$ | $y_b$, in. | $Ay_b$, in.$^3$ | $I$, in.$^4$ | $d$, in. | $Ad^2$, in.$^4$ | $I + Ad^2$, in.$^4$ |
|---|---|---|---|---|---|---|---|
| Girder | 1105.00 | 40.39 | 44,630.95 | 935,586 | 18.93 | 395,971 | 1,331,557 |
| Haunch | 78.00 | 79.00 | 6162.00 | 26.00 | 19.68 | 30,210 | 30,236 |
| Slab | 785.18 | 84.00 | 65,955.12 | 4187.62 | 24.68 | 478,255 | 482,453 |
| $\Sigma$ | 1968.20 | n.a. | 116,749.75 | n.a. | n.a. | n.a. | 1,844,236 |

Note: $A$ = cross-sectional area of component; $d$ = vertical distance from centroid of component area to centroid of composite section; $I$ = moment of inertia of component; n.a. = not applicable; $y_b$ = vertical distance between bottom of composite section and centroid of component; $\Sigma$. 1 in. = 25.4 mm; 1 in.$^2$ = 645 mm$^2$; 1 in.$^3$ = 16,387 mm$^3$; 1 in.$^4$ = 416,231 mm$^4$.

$y_{bc}$ = vertical distance between bottom of composite section and centroid of gross composite section = 116,749.75/1968.20

= 59.32 in. (1507 mm)

$e_{pc}$ = 59.32 – 7.46 = 51.86 in. (1317 mm)

## Deck-slab self-weight moment $M_d$

Girder span at bridge = 129.8 ft (39.56 m)

$$M_d = \frac{\left\{(12.583)(8/12) + \left[(60)(2)/144\right]\right\}(150)(129.8)^2}{(8)(1000)} = 2913.24 \text{ kip-ft (3950 kN-m)}$$

## Deck-slab section properties

$A_d$ = gross cross-sectional area of deck concrete (including haunch) = (12.583)(12)(8) + (60)(2) = 1328 in.$^2$ (856,772 mm$^2$)

$V/S$ = 1328/[12.583(12) + 2(2) + (12.583(12) – 60)] = 5.40

$e_d$ = eccentricity of strands with respect to centroid of transformed gross composite section = 78 + 2 + (8/2) – 59.32 = 24.68 in. (627 mm)

## Elastic shortening loss per article 5.9.5.2.3

### Elastic shortening loss using Eq. (C5.9.5.2.3a-1)

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_{pg}^2 A_g\right) - e_{pg} M_g A_g}{A_{ps}\left(I_g + e_{pg}^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}$$

$$\Delta f_{pES} = \frac{(12.152)(202.5)\left[935,586 + (32.93)^2(1105)\right] - (32.93)(30,524)(1105)}{12.152\left[935,586 + (32.93)^2(1105)\right] + \frac{(1105)(935,586)(4951)}{28,000}} = 19.835 \text{ ksi (137 MPa)}$$

$f_{pat}$ = stress in prestressing steel immediately after transfer

$= f_{pbt} - \Delta f_{pES}$ = 202.500 – 19.835 = 182.665 ksi (1259 MPa)

### Confirm $\Delta f_{pES}$ loss using Eq. (5.9.5.2.3a-1)

$$f_{cgp}(girder) = \frac{M_g e_{pg}}{I_g} = \frac{(30,524)(32.93)}{935,586} = 1.074 \text{ ksi (7 MPa)}$$

$$f_{cgp}(prestress) = \frac{f_{pat} A_{ps}}{A_g} = \frac{f_{pat} A_{ps} e_{pg}^2}{I_g}$$

$$= \frac{(202.5 - 19.835)(12.152)}{1105} + \frac{(202.5 - 19.835)(12.152)(32.93)^2}{935,586} = 4.582 \text{ ksi (32 MPa)}$$

$f_{cgp}$ $= f_{cgp}(prestress) - f_{cgp}(girder)$ = 3.507 ksi (24 MPa)

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} = \frac{28,000}{4951}(3.507) = 19.834 \text{ ksi (137 MPa)} \rightarrow \text{Confirmed}$$

## Approximate estimate of time-dependent losses per article 5.9.5.3

$$\Delta f_{pLT} = 10.0\frac{f_{pbt}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

where

$\gamma_h$ = correction factor for relative humidity of ambient air

= $1.7 - 0.01H = 1.7 - (0.01)(75) = 0.95$

$H$ = average annual ambient relative humidity = 75%

$\gamma_{st}$ = correction factor for specified concrete strength at time of prestress transfer

$$= \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 6.670} = 0.65$$

$\Delta f_{pR}$ = prestress loss due to relaxation of prestressing strands = 0 (typically 2.4 ksi for low-relaxation strand but taken as zero for comparison with measured losses)

$$\Delta f_{pLT} = (10.0)\left[\frac{(202.5)(12.152)}{1105}\right](0.95)(0.65) + (12.0)(0.95)(0.65) + 0 = 21.161 \text{ ksi (146 MPa)}$$

## Refined estimate of time-dependent losses per article 5.9.5.4

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}) + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})$$

### Shrinkage loss between transfer and deck placement per article 5.9.5.4.2a

$$\Delta f_{pSR} = \varepsilon_{bid}E_pK_{id}$$

where

$K_{id}$ = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in section being considered for time period between transfer and deck placement

$\varepsilon_{bid}$ = girder concrete shrinkage strain between transfer and deck placement

$$K_{id} = \frac{1}{1 + \left(\frac{E_pA_{ps}}{E_{ci}A_g}\right)\left(1 + \frac{A_ge^2_{pg}}{I_g}\right)\left[1 + 0.7\Psi_b(t_f, t_i)\right]}$$

where

$$\Psi_b(t_f, t_i) = 1.9k_sk_{hc}k_fk_{td}t_i^{-0.118}$$

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

$= 1.45 - 0.13(V/S) \geq 1.0 \rightarrow$ Defaults to 1.0 when $V/S \geq 3.5$

$k_{hc}$ = humidity factor for creep

$= 1.56 - 0.008H = 1.56 - 0.008(75) = 0.96$

$k_f$ = factor for effect of concrete strength

$$= \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 6.670} = 0.65$$

$k_{td}$ = time development factor

$$k_{td}(final) = \left(\frac{t}{61 - 4f'_{ci} + t}\right) = \left(\frac{20,000}{61 - 4\left(6.670\right) + 20,000}\right) = 1.00 \rightarrow$$

Note: final time age assumed to be 20,000 days (~50 years).

$t$ = age of concrete

$t_i$ = age of concrete at time of load application (release) = 1 day

$\Psi_b(t_f, t_i) = (1.9)(1.0)(0.96)(0.65)(1.0)(1^{-0.118}) = 1.19$

$$K_{id} = \frac{1}{1 + \left[\frac{\left(28,000\right)\left(12.152\right)}{\left(4951\right)\left(1105\right)}\right]\left[1 + \frac{\left(1105\right)\left(32.93\right)^2}{935,586}\right]\left[1 + 0.7\left(1.19\right)\right]} = 0.79$$

$k_{hs}$ = humidity factor for shrinkage $= 2.00 - 0.014H = 2.00 - 0.014(75) = 0.95$

$$k_{td}(deck) = \left(\frac{t}{61 - 4f'_{ci} + t}\right) = \left(\frac{185}{61 - 4\left(6.670\right) + 185}\right) = 0.84 \rightarrow$$

Note: deck slab was added 185 days after strand release.

$\varepsilon_{bid} = k_s k_{hs} k_f k_{td}(deck)(0.48 \times 10^{-3}) = (1.0)(0.95)(0.65)(0.84)(0.48 \times 10^{-3}) = 0.000249$

$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} = (0.000249)(28,000)(0.79) = 5.508$ ksi (38 MPa)

## Creep loss between transfer and deck placement per article 5.9.5.4.2b

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \Psi_b\left(t_d, t_i\right) K_{id}$$

$\Psi_b(t_d, t_i) = (1.9)(1.0)(0.96)(0.65)(0.84)(1^{-0.118}) = 1.00$

$$\Delta f_{pCR} = \left(\frac{28,000}{4951}\right)(3.507)(1.00)(0.79) = 15.669 \text{ ksi (108 MPa)}$$

## Relaxation loss between transfer and deck placement per article 5.9.5.3.2c

$\Delta f_{pR1}$ = 0 (typically 1.2 ksi for low-relaxation strand but taken as zero for comparison with measured losses)

## Shrinkage loss between deck placement and final time per article 5.9.5.4.3a

$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$

where

$K_{df}$ = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in section being considered for time period between deck placement and final time

$$= \frac{1}{1 + \left(\frac{E_p A_{ps}}{E_{ci} A_c}\right)\left(1 + \frac{A_c e_{pc}^2}{I_c}\right)\left[1 + 0.7\boldsymbol{\Psi}_b\left(t_f, t_i\right)\right]}$$

$$= \frac{1}{1 + \left[\frac{(28,000)(12.152)}{(4951)(1968)}\right]\left[1 + \frac{(1968)(51.86)^2}{1,844,236}\right]\left[1 + (0.7)(1.19)\right]} = 0.80$$

$\varepsilon_{bdf}$ = girder concrete shrinkage strain between time of deck placement and final time = $\varepsilon_{bif} - \varepsilon_{bid}$

$\varepsilon_{bif}$ = girder concrete shrinkage strain between placement and final time

= $k_s k_{hs} k_f k_{td}(final)(0.48 \times 10^{-3}) = (1.0)(0.95)(0.65)(1.0)(0.48 \times 10^{-3}) = 0.000296$

$\varepsilon_{bdf}$ = 0.000296 – 0.000249 = 0.000047

$\Delta f_{pSD}$ = (0.000047)(28,000)(0.80) = 1.053 ksi (7 MPa)

## Creep loss between deck placement and final time per article 5.9.5.4.3b

$\Delta f_{pCD} = \Delta f_{pCD1} + \Delta f_{pCD2}$

where

$\Delta f_{pCD1}$ = prestress loss due to creep of girder concrete caused by initial loads (prestress + self-weight)

$$= \frac{E_p}{E_{ci}} f_{cgp}\left[\boldsymbol{\Psi}_b\left(t_f, t_i\right) - \boldsymbol{\Psi}_b\left(t_d, t_i\right)\right] K_{df}$$

$$= \left(\frac{28,000}{4951}\right)(3.507)(1.19 - 1.00)(0.80) = 3.015 \text{ ksi (21 MPa)}$$

$\Delta f_{pCD2}$ = prestress gain due to creep of girder concrete caused by forces induced after initial loading (deck slab weight + additional dead load)

$$= \left(\frac{E_p}{E_c}\right)\Delta f_{cd}\boldsymbol{\Psi}_b\left(t_f, t_d\right) K_{df}$$

where

$\Psi_b(t_f, t_d)$ = girder creep coefficient at final time due to loading introduced at deck placement; creep coefficient of deck concrete at final time due to loading introduced shortly after deck placement

$$\Delta f_{pcd} = -\left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)\left(\frac{A_{ps}}{A_g}\right)\left(1 + \frac{A_g e_{pg}^2}{I_g}\right) - \left(\frac{M_d e_{ptg}}{I_{tg}}\right) - \left(\frac{M_{SIDL} e_{ptc}}{I_{tc}}\right)$$

where

$e_{ptc}$ = eccentricity of strands with respect to centroid of transformed composite section

$I_{tc}$ = moment of inertia of composite section calculated using transformed concrete section properties and the deck-to-girder modular ratio at service

$I_{tg}$ = moment of inertia of transformed girder section at service

$M_{SIDL}$ = maximum positive moment due to superimposed dead load

For span 43 of the Rigolets Bridge, $M_{SIDL}$ is negligible and therefore was not considered in the previous calculation.

$$\Delta f_{cd} = -\left(5.508 + 15\,669 + 1.200\right)\left(\frac{12.152}{1105}\right)\left[1 + \frac{\left(1105\right)\left(32.93\right)^2}{935,586}\right] - \left[\frac{\left(34,959\right)\left(31.67\right)}{981,451}\right]$$

$= –1.689$ ksi ($–12$ MPa)

$\Delta f_{pR1}$ loss needs to be included in the $\Delta f_{cd}$ equation even though it is not included in the summation of loss components.

$\Psi_b(t_f, t_d) = (1.9)(1.0)(0.96)(0.65)(1.0)(185^{-0.118}) = 0.64$

$$\Delta f_{pCD2} = \left(\frac{28,000}{6062}\right)(-1.689)(0.64)(0.80) = -3.994 \text{ ksi } (-28 \text{ MPa})$$

$$\Delta f_{pCD} = 3.015 + (-3.994) = -0.979 \text{ ksi } (-7 \text{ MPa})$$

## Relaxation loss between deck placement and final time per article 5.9.5.4.3c

$\Delta f_{pR2} = 0$ (typically 1.2 ksi for low-relaxation strand but taken as zero for comparison with measured losses)

## Shrinkage of deck concrete per article 5.9.5.4.3d

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1 + 0.7\Psi_b\left(t_f, t_d\right)\right]$$

where

$\Delta f_{cdf}$ = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete

$$= \frac{\varepsilon_{ddf} A_d E_{cd}}{\left[1 + 0.7\Psi_d\left(t_f, t_d\right)\right]}\left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right)$$

where

$\varepsilon_{ddf}$ = deck slab concrete shrinkage strain between placement and final time

$\Psi_d(t_f, t_d)$ = deck creep coefficient at final time due to loading introduced at deck placement

$= 1.9k_s k_{hc} k_f k_{td} t_i^{-0.118}$

$k_s$ = 1.45 – 0.13($V/S$) ≥ 1.0 → Defaults to 1.0 when $V/S$ ≥ 3.5

$k_{hc}$ = 1.56 – 0.008$H$ = 1.56 – (0.008)(75) = 0.96

$$k_f = \frac{5}{1+f'_{ci}} = \frac{5}{1+(0.80)(4.200)} = 1.15$$

$$k_{td}(final) = \left(\frac{t}{61-4f'_{ci}+t}\right) = \left[\frac{(20,000-185)}{61-4(4.200)+(20,000-185)}\right] = 1.00 \rightarrow$$

Note: final time age assumed to be 20,000 days (~50 years) minus 185 days (age of deck cast).

$t_i$ = 1 day

$\Psi_d(t_f, t_d)$ = (1.9)(1.0)(0.96)(1.15)(1.0)(1-$^{0.118}$) = 2.09

$\varepsilon_{ddf}$ = $k_s k_{hs} k_f k_{td}(0.48 \times 10^{-3})$ = (1.0)(0.95)(1.15)(1.0)(0.48 × $10^{-3}$) = 0.000524

$$\Delta f_{cdf} = \frac{(0.000524)(1328)(3929)}{[1+(0.7)(2.09)]}\left[\frac{1}{1968} - \frac{(51.86)(24.68)}{1,844,247}\right] = \text{-0.206 ksi (-1 MPa)}$$

$$\Delta f_{pSS} = \frac{28,000}{6062}\text{(-0.206)(0.80)[1 + 0.7(0.64)] = -1.102 ksi (-8 MPa)}$$

## Total time-dependent losses per Eq. 5.9.5.4.1-1

$\Delta f_{pLT}$ = (5.508 + 15.669 + 0) + [1.053 + (-0.979) + 0 + (-1.102)] = 20.149 ksi (139 MPa)

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## Synopsis


One span of the Rigolets Pass Bridge containing four 131-ft-long (40 m), high-strength concrete bulb-tee girders was instrumented and monitored to obtain measured strain and temperature data. During fabrication of the four girders for span 43 of the Rigolets Pass Bridge, several vibrating-wire strain gauges were installed near the midspan. During construction of the high-performance concrete deck slab for span 43, additional instrumentation was installed to measure concrete strains and temperatures. Samples of the girder and deck slab concrete were obtained and used for material property studies.

Throughout the bridge construction process, instrumentation readings were taken and recorded at selected time- or event-based intervals. After completion of the Rigolets Pass Bridge construction, the instrumentation installed in the girders and deck slab of span 43 was connected to an automated, on-site data acquisition system with remote-access capabilities. The on-site data acquisition system was used to record concrete temperature and strain data at hourly intervals for a period of one year after completion of construction.

Measured girder prestress losses derived from concrete strains corrected for temperature and load effects were less than corresponding values calculated using *AASHTO LRFD Bridge Design Specifications*. Prestress losses calculated using both the approximate and refined estimates of time-dependent losses were evaluated relative to the measured losses.



## Keywords

Bulb-tee girder, high-strength concrete, prestress loss, vibrating-wire strain gauge.


## Review policy

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

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