Comparison of Methods to Estimate Long-Term Deflections for a Precast, Prestressed Concrete Bridge Beam with a Cast-in-Place Composite Deck

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Estimating camber and deflection in precast, prestressed concrete bridge elements is a common topic of confusion and disagreement among design engineers, contractors, fabricators, and other stakeholders. In many design provisions and specifications, a net-positive design camber is required after all dead loads are placed. In other words, the girder is not supposed to sag under dead loads: self-weight, deck weight, and superimposed dead loads (barriers, diaphragms, and future wearing surface). Some agencies require a very specific net-positive design camber, such as ½ in. minimum.

Given that camber estimates in design may vary by 50%, how is this accuracy achieved in the final design? Is this an elastic requirement, or is it a time-dependent requirement at 20,000 days or greater based on service life? If the latter, how does a design engineer forecast the effects of creep, shrinkage, and relaxation on the deflection of the system? Can prestress losses really be determined accurately enough or with any level of precision for this final requirement?

Section 8.7 of the *PCI Bridge Design Manual*[1] presents an approximate method for estimating camber that uses a simple constant multiplier method originally proposed by Martin[2] in 1977. However, the basis of these multipliers, shown in Table 8.7.1-1, was never intended for bridge members made composite with a cast-in-place deck. This point is noted in Section 8.7:

> *This method gives reasonable estimates for cambers at the time of erection. The method does not, however, properly account for the significant effects of a large cast-in-place deck. The presence of a deck, once cured, drastically changes the stiffness of a typical bridge member. This has the effect of restraining the beam creep strains that are the result of prestressing, member self weight, and the dead load of the deck itself. Also, differential creep and shrinkage between the precast beam and the cast-in-place concrete can produce changes in member deformation. The multipliers for long-term deflection suggested by this method, therefore, should not be used for bridge beams with structurally composite cast-in-place decks.*
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> *In addition, it is not recommended that prestressing levels be increased in order to reduce or eliminate long-term downward deflection that might be predicted if the multipliers in Table 8.7.1-1 are used.*

Table 8.7.1-1 Suggested multipliers to be used as a guide in estimating long-term cambers and deflections for typical members

		Without Composite Topping	With Composite Topping
	At erection:		
(1)	Deflection (↓) component – apply to the elastic deflection due to the member weight at transfer of prestress	1.85	1.85
(2)	Camber (↑) component – apply to the elastic camber due to prestress at the time of transfer of prestress	1.80	1.80
	Final:		
(3)	Deflection (↓) component – apply to the elastic deflection due to the member weight at transfer of prestress	2.70	2.40
(4)	Camber (↑) component – apply to the elastic camber due to prestress at the time of transfer of prestress	2.45	2.20
(5)	Deflection (↓) component – apply to elastic deflection due to superimposed dead load only	3.00	3.00
(6)	Deflection (↓) component – apply to elastic deflection caused by the composite topping	---	2.30

Source: Reproduced by permission from *PCI Bridge Design Manual*, 3rd ed.

The Washington State Department of Transportation's *Bridge Design Manual (LRFD)*[3] further explains field observations of long-term camber in Section 5.2.4.B.8:

> *It might be expected that the above deck slab dead load deflection would be accompanied by a continuing downward deflection due to creep. However, many measurements of actual structure deflections have shown that once the deck slab is poured, the girder tends to act as though it is locked in position.*

Nevertheless, most design software programs use the long-term or "final" multipliers given in Table 8.7.1-1 of the *PCI Bridge Design Manual* as default multipliers for estimating final deflections.

Design Impact

Engineers who add strands to girder designs to achieve net-positive camber under all dead loads and after all losses by using the PCI deflection multipliers may find their designs can be conservative in terms of service stresses in the bottom fiber—that is, there may be more precompression in the bottom fiber than is required—because this method overestimates the time-dependent dead load deflection after the deck has cured. However, adding prestress to induce a net-positive camber can cause excessive tension in the top fiber near the ends, and addressing that issue may require a revised design with debonded or harped strands. Such a conservative design for the service limit state may also make lifting, handling, and transportation of the beam more difficult because of elevated tension in the top flange. Furthermore, adding strands to avoid a negative final camber can cause excessive camber in storage and at erection, which may cause issues in the field.

Materials

The constituents of today's high-performance concrete accompanied by higher transfer strengths, low water-cement ratios, and requirements on some projects for precast concrete beams to have a minimum age before casting the deck, all reduce creep effects on the deformation of the composite system. The original PCI multipliers were formulated using an ultimate creep coefficient of 2.0 in accordance with the 1971 edition of the American Concrete Institute's *Building Code Requirements for Reinforced Concrete* (ACI 318-71)[4] and considering the recommendations in *Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete* (ACI 209)[5] for concrete elements without compressive reinforcement and approximations for creep and shrinkage in a precast concrete element. Since that time, significant material advancements, such as implementation of ultra-high-performance concrete and improved curing methods, have further reduced long-term creep effects.

Alternative Design Method and Results

Analysis methods that are more rigorous than the simple constant multiplier method are now available to more accurately estimate the long-term deflection of a bridge system. These alternative methods are commonly employed in complex projects such as concrete segmental construction.

For the standard bridge with a cast-in-place concrete deck, Section 8.13.2.2 of the *PCI Bridge Design Manual* describes a time-dependent method based on information published in several books and articles such as the 1982 article by Dilger.[6] This method is a sectional analysis using creep, transformed section properties, and curvature for each time step and loading condition.[7,8] The basic time steps are as follows:

1. Transfer of prestress (instantaneous)
2. From transfer of prestress to deck placement (time dependent)
3. Placement of deck and superimposed dead load (instantaneous)
4. From placement of deck and superimposed dead load to final (time dependent)

The instantaneous stresses and strains are computed using transformed section properties with the material properties at the time of each analysis step. The time-dependent stresses and strains within a particular time step are computed as follows:

1. Calculate the age-adjusted effective modulus and corresponding modular ratio for the steel and the transformed composite section properties.
2. Determine the free strains that develop during the time interval being considered due to shrinkage, and those due to creep resulting from stresses applied during previous time steps.
3. Determine the artificial restraint of stresses and curvatures and the resultant normal force and moment at the centroid of the transformed section to prevent occurrence of the free strains in the previous step.
4. Apply these forces in the reverse direction to the centroid of the age-adjusted transformed section and calculate the time-dependent strains and stresses in the section.
5. Sum the results of steps 2 and 4 to provide net strains and stresses at each element corresponding to the restraining effect of the prestressing steel.
6. Determine deflections using standard formulas and the time-dependent curvatures computed as outlined in step 4 for the beam end and midspan.

This method of analysis was used to compute multipliers for various depths and selected spacings of precast concrete bulb-tee shapes common to the Mid-Atlantic and Northeast regions (**Fig. 1**).

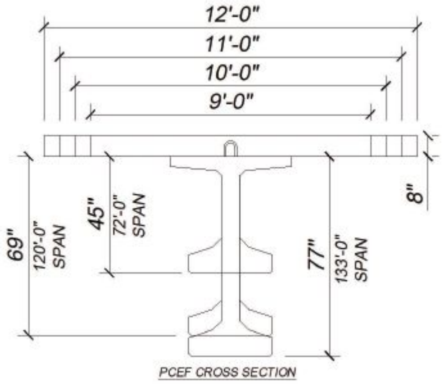

Figure 1. Precast concrete economical fabrication (PCEF) bulb-tee cross sections analyzed in this study. All Figures: JP Binard and Al Patel.

The effective deck thicknesses used in the analysis were 8 in. for 9- and 10-ft beam spacings and 8.5 in. for 10- and 11-ft spacings. The concrete design compressive strengths were 6.4 ksi at transfer, 8 ksi at 28 days, and a 4-ksi deck concrete compressive strength. The deck placement was assumed to take place at 56 days and the final period was 27,000 days or about 75 years.

We also investigated a historical example of an AASHTO Type III beam (62 ft 0 in. span) and Type IV beam (85 ft 0 in. span) using, respectively, a concrete compressive strength at transfer of 4 ksi and a design compressive strength at 28-days of 5 ksi, and a concrete compressive strength at transfer of 4.8 ksi and a design compressive strength at 28-days of 6 ksi. Such material strengths were common in the 1980s and might more closely resemble the conditions for which the PCI multipliers were developed. The deck concrete strength was assumed to be 4 ksi for these cases, and the deck placement was assumed to take place at 56 days with the final period of 27,000 days or about 75 years.

Lastly, we included a stretched span-to-depth design using a shallow 37-in.-deep bulb tee with a spacing of 9 ft 0 in. and a span length of 90 ft 0 in.

Figures 2 and **3** compare the results of the calculated long-term prestress multiplier (upward camber) and self-weight multiplier (downward), respectively, for the different scenarios investigated.

The prestress and self-weight multipliers calculated with a lower compressive strength at transfer and the smaller cross

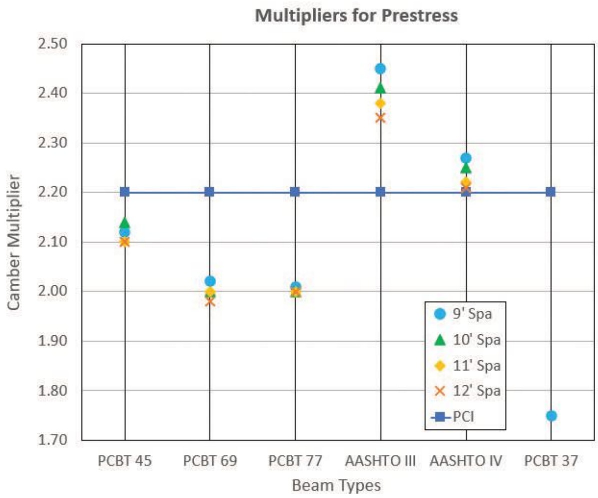

Figure 2. Long-term multiplier to be applied to elastic camber at time of transfer due to prestress for several girder configurations computed using PCI multipliers and a more refined method from the *PCI Bridge Design Manual.*

Figure 3. Long-term multiplier to be applied to elastic self-weight deflection at time of transfer for several girder depths and spacings computed using PCI multipliers and a more refined method from *PCI Bridge Design Manual.*

section (AASHTO Type III) more closely follow the PCI multipliers, whereas the longer-span bulb tees with higher transfer strengths do not.

A key variable in this analysis is the magnitude of deck shrinkage to be considered. Section 8.6.7.3 of the *PCI Bridge Design Manual* recommends that only 50% of the deck shrinkage be applied, but the *AASHTO LRFD Bridge Design Specifications*[9] do not specify a percentage. Therefore, we compared results of calculations made with the PCI multipliers using both 100% and 50% of the deck shrinkage along with the deck weight up to the final time step (**Fig. 4** and **5**). In general, for the cases based on 100% of the deck shrinkage, the calculated multipliers were slightly lower than the PCI multiplier. The 45-in.-deep precast concrete bulb-tee (PCBT 45) beams were the exception, which is likely due to two main factors: the lower span-to-depth ratio as compared with the other beams, and the computed deck deflections with respect to span length. However, for the cases that used 50% of the deck shrinkage, the calculated multipliers were notably lower.

Fig. 6 compares superimposed dead-load multipliers for final conditions (after deck placement). In this case, all calculated multipliers are lower than the PCI multiplier.

As previously mentioned, we investigated a case with a shallow 37-in.-deep bulb-tee beam and a stretched span to study the long-term camber behavior for a high span-to-depth ratio. The estimated long-term deflection using PCI multipliers is a sag of 0.1 in. When using 50% of the deck shrinkage and the age-adjusted transformed properties, the result is a positive camber of 1.5 in. **Table 1** compares the calculated camber using the time-step method and the PCI multipliers.

Topics for Further Study

Direction from industry on the required percentage of deck shrinkage to apply when estimating long-term deflection in conventional bridge construction is important for greater accuracy in those estimates. Furthermore, current practice in some states is to design bridges as continuous for live loads with the continuity connection between spans made at a minimum specified time after transfer of prestress. Changing the system behavior from a simple span to a continuous span after the continuity connection has been made may result in smaller long-term deflection values than determined in this article.[10] Future study is required to refine the multipliers for a continuous system for the period after deck placement, which is when the

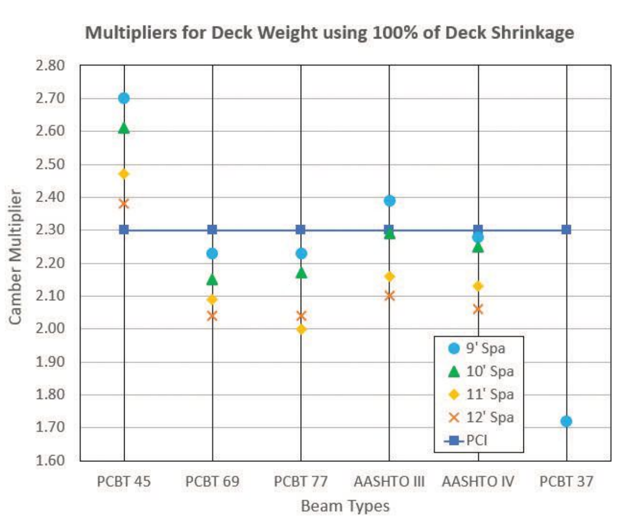

Figure 4. Long-term multiplier to be applied to elastic deck weight deflection, considering 100% of deck shrinkage at final for various girder depths and spacings computed using PCI multipliers and a more refined method from the *PCI Bridge Design Manual.*

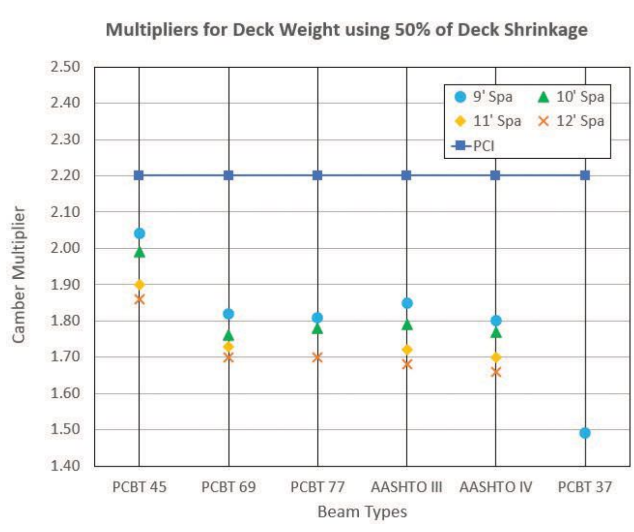

Figure 5. Long-term multiplier to be applied to elastic deck weight deflection, considering 50% of deck shrinkage at final for various girder depths and spacings computed using PCI multipliers and a more refined method from the *PCI Bridge Design Manual.*

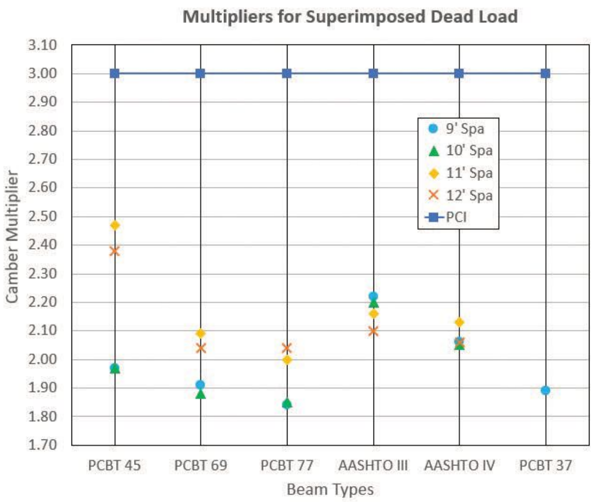

Figure 6. Superimposed dead-load multiplier for several girder depths and spacings computed using PCI multipliers and a more refined method from the *PCI Bridge Design Manual*.

Table 1. Comparison of long-term multipliers and computed design deflections using 50% of deck shrinkage for a 37-in.-deep bulb-tee girder spanning 90 ft 0 in. with a 9 ft 0 in. girder spacing

Long-term multiplier comparison					
Parameter	Elastic deflection, in.	Detailed analysis		PCI multipliers	
		Multiplier	Total deflection, in.	Multiplier	Total deflection, in.
Prestress	5.70	1.74	9.90	2.20	12.50
Self-weight	-1.80	1.78	-3.20	2.40	-4.40
Deck, 50%	-3.30	1.48	-4.90	2.30	-7.60
Super-imposed dead load	-0.18	1.89	-0.34	3.00	-0.60
			1.46		-0.10

continuity connection is made. Additional time steps for the time-dependent effects between deck placement and continuity connection may be necessary to determine appropriate multipliers for final deformation.

Conclusion and Recommendations

Wide-flange bulb-tee beams, used in conjunction with increased concrete strengths and larger-diameter strands, have allowed efficient designs with increased span-to-depth ratios and beam spacings in bridges with cast-in-place decks. Furthermore, the advancement of material technologies such as ultra-high-performance concrete may require further refinement in analyzing deformations beyond common design assumptions based on tabulated multipliers. Therefore, because the accuracy of the long-term deflection estimates is important to most owners, a program using regional field data to create region-specific multipliers based on beam types used by local prestressed concrete producers, design and construction practices, and bridge history may be beneficial. Until those data are available or analyzed, multipliers may be computed for commonly produced prestressed concrete beams using the method presented in this article for both 50% and 100% of deck shrinkage to provide a reasonable range of values (**Table 2**).

The multiplier values developed in this study apply to a limited set of beam types, specific conditions, and certain assumptions. Although these multipliers, which are lower than standard PCI multipliers, may provide more accurate estimates of long-term camber that can possibly lead to more efficient designs, bridge engineers should only use them with a full understanding of their derivations, assumptions, and implications.

Table 2. Average multipliers for long-term effects and comparisons using 50% and 100% of deck shrinkage

Deck cast at 56 days using 50% of deck shrinkage					
	PCBT	AASHTO types	All beams	PCI	Recommended
Prestress	2.02	2.32	2.14	2.20	2.10
Member self-weight	1.93	2.08	1.99	2.40	2.00
Deck weight	1.81	1.75	1.79	2.30	1.90
Superimposed dead load	1.93	2.12	2.00	3.00	2.00

Deck cast at 56 days using 100% of deck shrinkage					
	PCBT	AASHTO types	All beams	PCI	Recommended
Prestress	2.02	2.32	2.14	2.20	2.10
Member self-weight	1.93	2.08	1.99	2.40	2.00
Deck weight	2.25	2.21	2.23	2.30	2.25
Superimposed dead load	1.93	2.12	2.00	3.00	2.00

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