THE DILEMMA OF PRESTRESSED GIRDER CAMBER VARIABILITY

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ABSTRACT

It has been common knowledge that the precast prestressed girder camber at prestress release and at time of erection can vary significantly. This happens even in cases where two identically prestressed girders are stored in the same manner and erected at the same time. The variations become more significant as the use of high strength concrete, longer spans, and more heavily prestressed girders continue to increase. Cambers as large as 8 inches with 3 inches variability are not uncommon. This problem may not be a safety issue, but it has created challenges for designers, owners, and contractors. This paper addresses several issues related to prediction, design, and construction to accommodate variability in girder camber:

(a) In design, it is impossible to precisely predict camber. However, if modern methods for calculating modulus of elasticity, creep, and prestress loss are used, the error in estimating the mean camber should be reduced and the most probable range of cambers can be predicted.

(b) Recommendations for determining final girder seat elevations and detailing of the composite action reinforcement can be made to accommodate the predicted camber and its variability.

(c) Often a point of contention between the owner and the contractor is the costs incurred for concrete shims over the girder flanges that are thicker than designed in order to accommodate cambers larger than predicted. This issue will be discussed.

This paper will include a detailed design example and the spreadsheet used in its calculations. The spreadsheet can be used as a design tool for other examples. Discussion in this paper is limited to conditions up to and including application of the deck slab weight.

Keywords: Camber, Deflection, Variability, Girder, Initial, Long-term.

INTRODUCTION

Camber variability is one of the most widely discussed issues during design, fabrication, and construction. Designers often use simplified calculations available in commercial software. Such calculations may be out of date or even theoretically questionable. The logic for their use is that since camber is a random variable anyway, it does not warrant a theoretically rigorous prediction. However, random variability of camber should not justify lack of rigor in prediction. It is not a good design practice to almost always over- or under-predict camber. What is predicted should be a good mean (or average) value, preferably with an indication of the range of variability.

In recent years, concrete strength at prestress release has increased from 4500 psi to 6500 psi and in some cases as high as 10,000 psi. Recent work by FHWA and several research agencies has produced concrete release strengths over 12,000 psi. This high strength allows for the use of relatively slender girders with more prestressing. Thus, camber can be expected to be higher than in girders with lower strength concrete. An offsetting effect is the higher stiffness of stronger concrete. This is represented by higher elasticity modulus and lower creep and shrinkage coefficients.

Although camber at release is not impacted by creep and shrinkage estimates, it is highly influenced by the elasticity modulus. Also, accurate estimates of elastic shortening losses at prestress release would allow for more accurate prediction of camber at release.

However, camber at the time of deck placement for composite girders is influenced by creep, shrinkage, and long term prestress losses. Long-term camber/deflection after the deck becomes composite with the precast girders has been shown by some software to be significant. In reality, however, the much higher stiffness of the composite system, the low differential deck/girder shrinkage and the relatively low girder creep after deck placement result in considerable stabilization of the camber beyond deck placement time. As will be demonstrated later, the multipliers used in these simplified calculations were originally developed for building double tees with a two inch concrete topping. It should also be noted that with high strength concrete increasing in use, most of the creep and shrinkage take place in the first few months of concrete age. Therefore, the previously assumed gradual development prediction formulas would not be accurate.

The 2005 Interim to AASHTO LRFD Specifications introduced extensive revisions to the prestress losses and to the modulus of elasticity, creep, and shrinkage formulas. They were introduced to extend the application of these formulas to concrete strengths from 5 to 15 ksi. This paper shows how these prediction formulas can be incorporated into a spreadsheet to calculate initial and long term camber. Results will be compared to results of existing methods.

Camber variability will be discussed. Recommendations will be made for user friendly detailing and construction methods to acknowledge camber variability and minimize conflicts between designers, producers, and contractors.

BACKGROUND AND METHODS OF INITIAL CAMBER ANALYSIS

Instantaneous camber, at the time of release of the prestressing force from the bed to the concrete member, appears to be straight forward. The prestressed concrete member cambers upward because the upward bending due to initial prestress is generally larger than the downward deflection due to member self weight. The camber at that time is a result of the combination of these two effects. Due to the assumed linear elastic behavior of the system, the conventional theory of elasticity and method of superposition are valid. Thus, deflection due to self weight is calculated separately from camber due to initial prestress, although the two quantities cannot be physically separated.

Textbooks on structural analysis contain formulas such as Equation 1, for the mid-span deflection, Δ_g, of a simply supported span subjected to a uniformly distributed load, w. The span length is L and the cross section rigidity is EI, where E is the modulus of elasticity and I is the moment of inertia.

$$\Delta_g = \frac{5WL^4}{384EI} \tag{1}$$

For camber due to initial prestress, P, Equation 2 is usually cited in the literature

$$\Delta_p = \frac{PL^2}{8EI}\left[e_c + (e_e - e_c)\frac{4b^2}{3L^2}\right] \tag{2}$$

where ec and ee are eccentricity of prestress relative to the centroid of the cross section at the center and at the end. The distance “b” is distance from the end of the member to the point of hold down. Equation 2 is valid for one-point depression, two-point depression and straight strands, by properly defining (ee-ec), and “b”.

Unlike structural steel, concrete is a material that is neither elastic nor time-independent. It is also not homogenous as it must contain reinforcement to function as a structural member. The assumption of a simple span supported on knife-edge supports with zero width and unrestrained rotational ability has conventionally been used with little significance on overall design and behavior prediction.

For camber analysis of prestress release, the following assumptions have historically been used in practice:

(a) The span length is assumed to equal the overall member length. The reasoning behind this assumption is that when prestress is released, the member cambers and the bottom of the girder separates from the bed except at the very ends. Some design guides use the span length between bearings on the actual bridge. This is done for convenience as will be illustrated later.

(b) The modulus of elasticity is the concrete modulus at time of prestress release. This quantity is most often predicted from the unit weight of the member and the specified concrete strength at prestress release.
(c) The prestress force is assumed to be the force in the concrete after allowance for elastic shortening losses. Shortening of the member as the prestress is transferred corresponds to two equal and opposite forces: tension in the strands and compression in the concrete. At this time, strand tension is smaller than the tension before release due to the member deformation.
(d) The properties e_c, e_e and I are the gross cross section properties. Theoretically, they should be the net section properties, as the calculation of elastic loss presumes separation of the steel and concrete. However, the two sets of properties are close.

A more rigorous approach would be to use the prestress force just before release, applied to transformed section properties, when calculating the initial camber. With this approach, it is not necessary to calculate the elastic shortening loss. As the elastic loss varies from one section to another along the span, and the error introduced by assuming it to be constant is avoided. This approach was introduced in the AASHTO LRFD Bridge Design Specifications for the first time in 2005. It is followed in the proposed equations later in this paper.

The use of draped strands is a common practice in concrete girder design. To further relieve excess prestress near the member ends, some of the strands are debonded (shielded, blanketed) for part of the length. Equation (2) does not take into account loss of prestress due to strand shielding. This effect will be included in the proposed formula.

As indicated earlier, the span length at this stage is usually assumed to be the full member length. This may be true during the short duration when the prestress is released and before the member is removed from the bed. However, when the member is stored in the precast yard, it is usually placed on hard wood blocking some distance in from the end. This condition remains until the member is shipped for erection on the bridge. It would seem more important to model the storage support condition rather than the short period the member is in the prestressing bed. Because of the increasing use of very long span girders, over 150 feet in length, it is always desirable to place the wood blocking at a distance close to 7 to 10% of the member length.

There is a need to standardize storage conditions in order to allow for more accurate camber prediction. At a minimum, the designer should recognize that support location during girder storage is a factor in estimating camber at release and at erection.

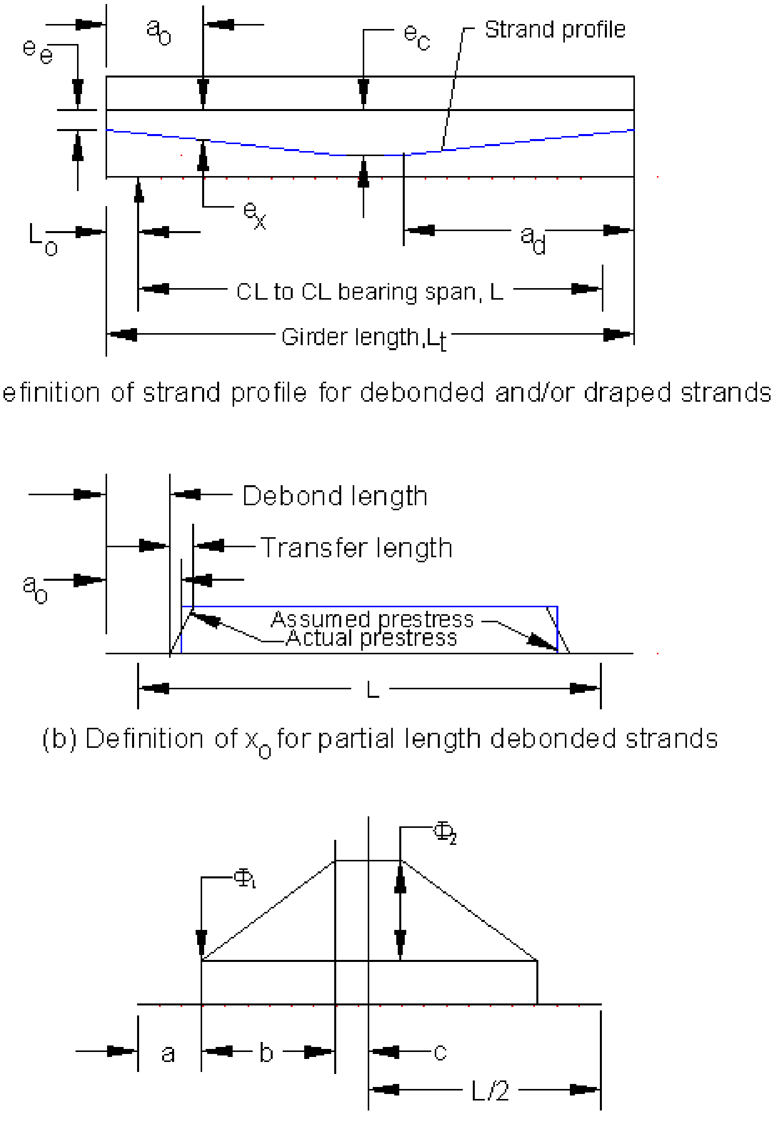
PROPOSED INITIAL CAMBER PREDICTION METHOD

A. INITIAL CAMBER DUE TO PRESTRESS

Consider the most general case of a group of strands that are draped at two points, and also debonded at the ends. Such a case, generally, does not exist in practice. However, one can use the same equation to calculate camber in the great majority of cases encountered in practice. For example, if the strands are draped and not debonded, the debond length is equal to zero. If the strands are debonded and not draped, then the eccentricity at the end is equal to eccentricity at mid-span. Further, the derivation takes into account that the girder may be placed on temporary supports at the yard which are several feet in from the end.

Referring to Fig. 1(a), the following geometric parameters are assumed to be known at various stages of the life of the member:

L = span length between supports
L_t = total member length
e_c = strand eccentricity at the center of the member measured from the centroid of the transformed section
e_e = strand eccentricity at end of the member measured from the centroid of the transformed section
a_d = distance from member end to hold down point
a_o = modified debond length = (actual debond length + transfer length/2).
L_o = overhang length, equal to $(L_t - L)/2$ (3)
$a = a_o - L_o$ (4)
$b = a_d - a_o$ (5)
$c = (L/2) - a - b$ (6)



(a) Definition of strand profile for debonded and/or draped strands

(b) Definition of x_o for partial length debonded strands

(c) Curvature Distribution due to initial prestress

Fig. 1 General modeling of pretensioning effects for initial camber

$$e_x = e_e + \frac{a_o}{a_d}(e_c - e_e) \tag{7}$$

Referring to Figure 1(c), the curvature and the theoretical end of the debond length:

$$\phi_1 = \frac{P_i e_x}{E_{ci} I_{ti}} \tag{8}$$

P_i = initial prestress force in the group of strands being considered, just before release to the concrete member

E_{ci} = modulus of elasticity of the girder concrete at time of prestress release.

I_{ti} = moment of inertia of precast transformed section at time of prestress release,

The modulus of elasticity of concrete, for normal weight aggregate concrete, at the time of prestress release can be obtained from the following formula in the AASHTO LRFD Specifications

$$E_{ci} = 33{,}000 K_1 w^{1.5} \sqrt{f'_{ci}} \qquad (9)$$

For normal weight aggregate concrete and in absence of more accurate information, the unit weight, w, may be estimated from AASHTO LRFD specifications:

$$0.145\text{kcf} \le w = (0.14 + 0.001 f'_c) \le 0.155\text{kcf} \qquad (10)$$

where
K_1 = a correction factor for source of aggregates, assumed =1.0 unless determined by testing.
f'_{ci} = specified concrete strength in ksi, at initial conditions
f'_c = specified concrete strength in ksi, at final service conditions, assumed in design to be at 28 days. The factor ($0.14+0.001\, f'_c$) is not to be taken less than 0.145 or greater than 0.155.
w = unit weight of concrete in (kcf)

The modulus of elasticity of concrete at service is also needed in calculating the instantaneous deflection due to deck weight and additional loads. Equation 9 can be used for this purpose with the term $\sqrt{f'_{ci}}$ replaced with $\sqrt{f'_c}$.
The NCHRP 18-07 project on which Equation (9) was based also includes a factor K_2 to account for the random variability of E_c. The factor K_2 varies from 0.82 to 1.2 for 10^{th} percentile lower-bound to 90^{th} percentile upper-bound values.

The curvature change, ϕ_2, due to the difference of eccentricity between the debond point and the harp point:

$$\phi_2 = \frac{P_i (e_c - e_x)}{E_{ci} I_{ti}} \qquad (11)$$

Integration of the curvature diagram gives the member slope. Integrating once more gives the member deflection. Integration is simple as the curvature diagram is a series of straight lines. The resulting mid-span camber can be shown to have the following formula:

$$\Delta_{ip} = \frac{\phi_1}{2}(b+c)(2a+b+c) + \frac{\phi_2}{6}(3ab + 2b^2 + 6ac + 6bc + 3c^2) \qquad (12)$$

Equation 12 can be simplified as

$$\Delta_{ip} = \frac{\phi_1}{8}(L^2 - 4a^2) + \frac{\phi_2}{6}(3ab + 2b^2 + 6ac + 6bc + 3c^2)$$

Equation 12 is general and applicable to the common cases encountered in practice. For example, for straight strands that are bonded full length, with the transfer length ignored, the initial camber, Δ_{ip}, due to prestress can be obtained from Equation (12) by setting $e_e = e_c$ and $a = b =$ zero.

Thus, $\phi_1 = \frac{P_i e_c}{E_{ci} I_{ti}}$, $\phi_2 = 0$, and

$$\Delta_{ip} = \frac{\phi_1}{2}(0 + c)(0 + 0 + c) + \frac{0}{6}(3ab + 2b^2 + 6ac + 6bc + 3c^2) = \frac{P_i e_c L^2}{8E_{ci} I_{ti}} \qquad (13)$$

which is a formula commonly encountered in literature.

Another common formula for strands with two-point draping, ignoring transfer length effects is as follows. The difference between the actual length and the span length can be obtained by setting a_o=L_o=a =0, and b+c = L/2

$$\Delta_{ip} = \frac{P_i L^2}{8E_{ci} I_{ti}}\left[e_c + (e_e - e_c)\frac{4b^2}{3L^2}\right] \qquad (14)$$

Often, a member contains some strands that are straight and full-length bonded, some that are straight and partial length bonded, and draped strands. Groups of strands of the same characteristics should be established. Camber due to each group is then calculated separately and the total camber due to prestress is obtained by simple summation.

B. INITIAL DEFLECTION DUE TO MEMBER WEIGHT

When the member is not supported at its ends, the overhangs create negative moments and also cause reduction in the midspan positive moment. The midspan deflection can be developed in terms of the moments at the ends and at midspan, using simple elastic analysis.

$$\Delta_{gi} = \frac{5L^2}{48E_{ci} I_{ti}}(0.1M_{e1} + M_c + 0.1M_{e2}) \qquad (15)$$

where

M_{e1}= moment at left support, negative if overhang exists, zero if overhang ignored

M_{e2}= moment at right support, negative if overhang exists, zero if overhang ignored

M_c = midspan moment

Most designers ignore the overhangs in estimating the initial deflection due to self weight. This may be reasonable for conventional beam lengths and for supports near the beam ends. However, in recent years, very long girders, approaching the 200 ft length, have been produced. These girders should be supported at a distance approximately 7 to 10% of the length. This will help improve stability, camber and sweep during storage. For such a condition, Equation (15) will give more accurate results than equations developed for a simple span.

As indicated above, the prestress force just before release along with section properties of the transformed section should be used in analysis. This is the method promoted by the AASHTO LRFD Specifications, Section 5.9.5. An alternative and historically common method has been to use gross section properties along with prestress force just after release, which is equal to P_i minus the elastic shortening loss. The method proposed here is the theoretically exact one as the assumption that the elastic loss is constant for the entire length is avoided. Also, the approximation that the gross section represents the true (net) concrete section does not have to be invoked. This latter approximation is insignificant, especially when relatively small prestress steel area is used. The first approximation, however, may underestimate the elastic loss effect and over-estimate camber.

EXAMPLE 1

Given: Florida 72" deep bulb tee. Cross section dimensions are shown in Figure 2. Gross cross section properties are A = 920.7 in^2, y_b = 34.05 in. and I = 655, 930 in^4. Overall girder length equals 137.083 ft, centerline to centerline bearing length equals 135.500 ft. Specified concrete strength for girder at release equals 6.0 ksi and at final 8.5 ksi. The prestressing consists of 44-0.6" strands tensioned to 202.5 ksi just before force is released to the concrete. All strands are straight. The strand pattern is 13 strands at 3 in. from the bottom face of the member, 13 at 5 in., 11 at 7 in., 5 at 9 in., 1 at 11in., and 1 at 13 in. There are 33 strands bonded full-length; 4 strands in the bottom row are debonded 14 ft at each end, 4 strands in the second row are debonded 8 ft and 3 in the third row are debonded 6 ft. The modulus of elasticity of the strands is assumed equal to 28,500 ksi.
Required**:** Camber prediction at prestress release.

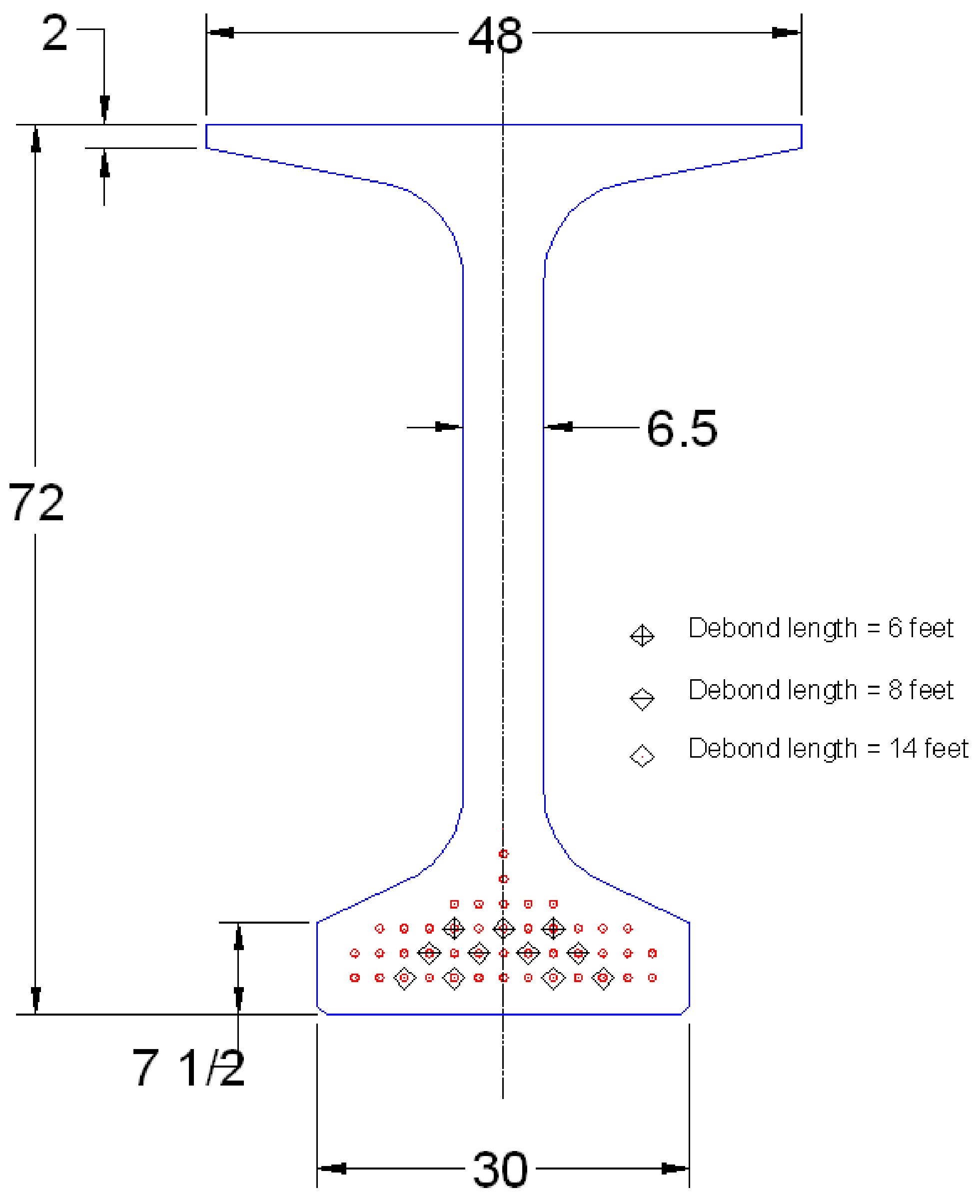


Fig. 2 Florida BT72 used in numerical example

Using Equation 9, the modulus of elasticity of the girder at release, based on minimum specified concrete strength,

The unit weight, w = 0.14+0.001*8.5 = 0.1485 kcf

$$E_{ci} = 33{,}000(1.0)(0.14 + 0.001 * 8.5)^{1.5}\sqrt{6} = 4626 \text{ ksi}$$

Also, the modulus of elasticity at time of deck placement, using concrete strength = 8.5 ksi is needed to calculate the elastic deflection due to deck weight. Similarly, it is calculated to be 5,506 ksi.

This value is used to calculate the transformed section properties. Midspan section properties are sufficiently accurate to use for the entire length, ignoring the effects of strand debonding and draping. The centroidal distance of the 44 strands relative to the bottom face of the member = (13*3+13*5+11*7+5*9+1*11+1*13)/44 = 5.68 in.

Modular ratio $n_i = \frac{E_{ps}}{E_{ci}} = \frac{28{,}500}{4{,}626} = 6.16$

Transformed area = $A_g + (n_i-1)\,A_{ps}$= 920.7+ (6.16-1)*44*0.217 = 970.0 in^2

Similarly, moment and inertia and centroidal depth of the transformed section at final can be calculated. These properties and the properties at the time of deck placement are summarized in the table below:

Table 1 Section properties of precast girder

	Gross	Transformed, initial	Transformed, final
A (in.^{2})	920.7	970.0	960.6
yb (in.)	34.05	32.61	32.87
I (in.^{4})	655930	693615	686723

The centroidal distance of the 33 fully bonded strands = (9*3+9*5+8*7+5*9+1*11+1*13)/33 = 5.97 in. The eccentricity of prestress relative to the centroid of the transformed section = 32.61-5.97 = 26.64 in. Similarly, the eccentricity of the groups of 4, 4, and 3 debonded strands can be shown to be 29.61, 27.61, and 25.61 in.

Initial Camber due to prestress:

The calculations can be programmed into a spreadsheet as given in the Appendix and downloadable from the website "www.structuresprograms.unomaha.edu".

Table 2 Strand grouping

Group 1	33 strands	Full length bonded-straight
Group 2	4 strands	Debonded 14 ft from ends-straight
Group 3	4 strands	Debonded 8 ft from ends-straight
Group 4	3 strands	Debonded 6 ft from ends-straight

For illustration, the procedure for camber calculation due to release of prestress in the second group of strand as shown in Table 3, will be demonstrated in detail below.

The initial prestressing force = $f_{pi}*A_{ps}$ = 202.5* 4*0.217 = 175.77 kips. The distance L_o from the end of the member to the support point = (137.083-135.5)/2 = 0.7915 ft. As shown in Figure 1, the distance "a" between support and the assumed start of prestress in girder = debond length + (transfer length/2) $-L_o$. Thus for these 4 strands, the value of "a" = 14+1.5-0.7915= 14.709 ft. Since the strands are not draped, any value for a_d, except zero, would work and is assumed here equal span/2 = 135.5/2 = 67.75. Zero would create an intermediate step mathematical error as it is in the denominator of an equation. With this information, one can calculate b = a_d-a_o = 52.3ft. Finally c = L/2-a-b = 135.5/2-14.709-53 = 0.741 ft. The strand eccentricity at the end of the debonded length, using Eq. 7, is 29.61 in. as expected since the strands are straight. Using Eq. (8), the curvature can be calculated as:

$\phi_1 = \dfrac{175.77*29.61}{4626*693615} = 1.62*10^{-6}$ in^{-1}. The curvature ϕ_2 is zero, as expected, since these are not draped strands. Finally, using Eq. 12, the camber due to this group of strands can be calculated as:

$$\Delta_{ip2} = \frac{1.62*10^{-6}}{2}(52.3+0.7915)(2*14.709+52.3+0.7915)*12^2 + \frac{0}{6}(3ab+2b^2+6ac+6bc+3c^2)$$
$$= 0.51 \text{ in.}$$

The calculations for the four groups of strands are summarized in Table 3.

Table 3 Initial camber due to prestress

Strand group	No. of strands	Force	L_o, ft	a_o	a	a_d	b	c	e_c	e_e	e_x	ϕ_1	ϕ_2	Δ_{ip}
		kips	ft	ft	ft	ft	ft	ft	in.	in.	in.	in^{-2}	in^{-2}	in.
1	33	1450.10	0.7915	1.5	0.7085	67.75	66.25	0.7915	26.6	26.6	26.6	1.2E-05	0	3.98
2	4	175.77	0.7915	15.5	14.709	67.75	52.25	0.7915	29.6	29.6	29.6	1.62E-06	0	0.51
3	4	175.77	0.7915	9.5	8.7085	67.75	58.25	0.7915	27.6	27.6	27.6	1.51E-06	0	0.49
4	3	131.83	0.7915	7.5	6.7085	67.75	60.25	0.7915	25.6	25.6	25.6	1.05E-06	0	0.34
		1933.47												5.33

Initial Camber due to member weight:

The girder is assumed to be placed on supports at the yard at the exact same locations as the permanent supports. In order to account for the support locations, the girder moments at the supports, M_{e1} and M_{e2} at midspan M_c need to be calculated. The unit weight of plain concrete has already been determined to be 0.1485 kcf. Allowing 5 lbs per cubic ft for the increase due to steel weight, the unit weight for load calculation is thus, 0.1485+0.005 = 0.1535 kcf. With concrete area = 920.7 in^2, the girder weight = (920.7)(0.1535)/144 = 0.9814 kip/ft.

M_{e1} = (0.9814* 0.7915^2/2)(12) = 3.69 kip-in.
M_c = (0.9814*135.5^2/8)(12) – 3.69 = 27025.55 kip-in

Thus

$$\Delta_{gi} = \frac{5(135.5)^2(12)^2}{48(4{,}626)(693{,}615)}(27025.55 - 0.1(3.69+3.69)) = 2.32 \text{ in.} \downarrow$$

Thus the net camber at release is estimated to be = 5.33-2.32 = 3.01 in. ↑

Initial camber predicted by CONSPAN, a popular commercial software, for the same example has camber 6.15 in due to prestress and deflection equal to 2.71 in due to member weight for a net camber of 3.44 in. With CONSPAN, the modulus of elasticity used was 0.9 of that predicted by AASHTO, to reflect FDOT design guide for soft Florida limerock aggregates. However, most of the difference, for this particular example, between the value predicted by the proposed method and that from CONSPAN can be attributed to the difference in modulus of elasticity. This will be further discussed in the next section.

VARIABILITY OF INITIAL CAMBER

There are number of causes of the variability of initial camber. Some of them are listed below.

RANDOM VARIABILITY OF CONCRETE MODULUS OF ELASTICITY

Studies by Tadros et al. (2003) have demonstrated that the values of E_{ci} can vary by $\pm 22\%$ relative to the mean value for levels of confidence between 10 percentile and 90 percentile. The scatter in the results from various sources was reported in Tadros et al. and is reproduced as Fig. 3. This variation alone would correspond to a range of camber for Example 1 girder from 0.78 to 1.22 of 3.01 in. The lower-bound value is thus 2.35 in. and the upper-bound value = 3.67 in.

IMPACT OF COARSE AGGREGATES OF E_{ci}

This is a well documented but often ignored factor. The Tadros et al. study gives recommendations for this effect for the states of Nebraska, Washington, Texas, and New Hampshire. In Florida, the use of soft native limerock is frequent enough to have a standard recommendation to use a 0.9 factor. If this factor is used, the camber changes from 3.01 in. to 3.01/0.9 =3.34 in. Because of these two factors, it is strongly recommended that records of the modulus of elasticity be kept for mixes used bridge girder production.

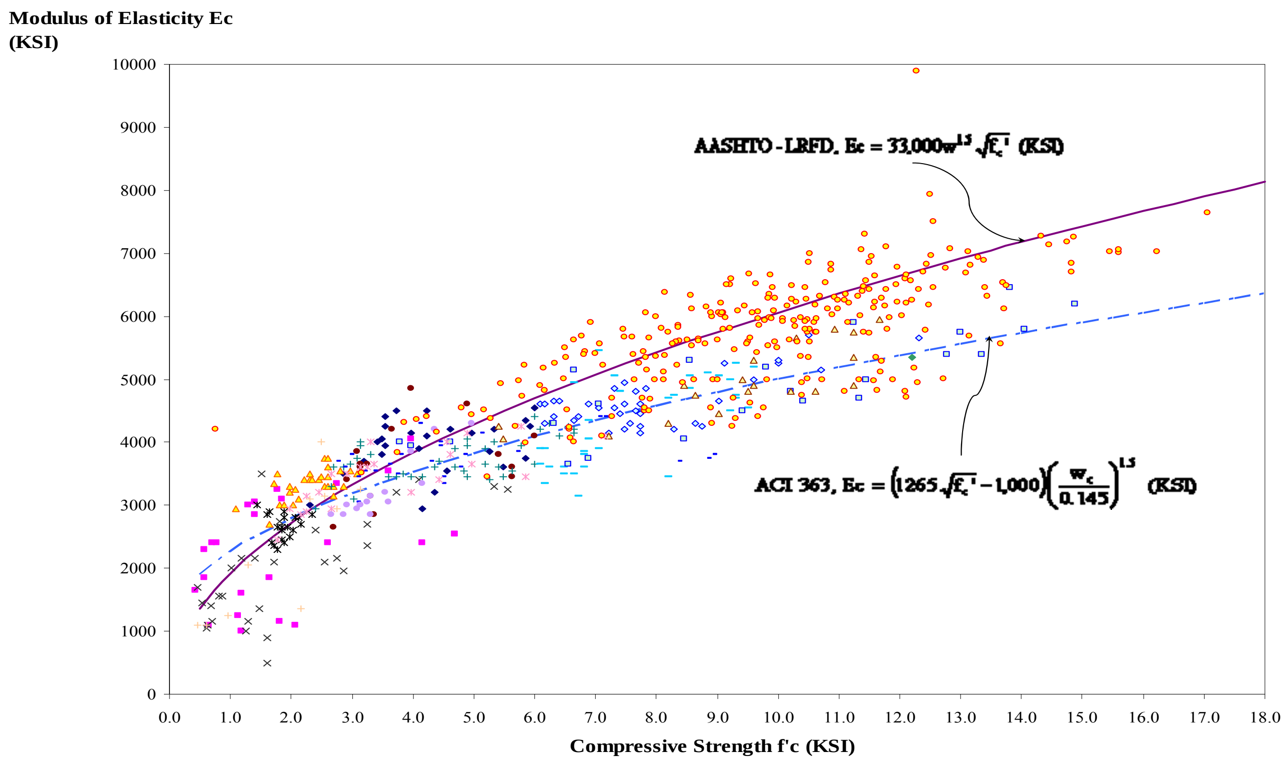


Fig. 3 Modulus of elasticity versus compressive strength as reported by Tadros et al (2003)

ACTUAL CONCRETE STRENGTH VERSUS SPECIFIED STRENGTH

Designers specify a minimum concrete strength at prestress release of, for example, 6 ksi. However, it is not an uncommon practice or a code violation to release the prestress at higher concrete strengths of 7 or 8 ksi. Occasionally, precasters leave girders scheduled for release on Saturday in the bed until Monday morning, two days later than the due date. Actual initial concrete strength can change dramatically in a short period, depending on the mix and is generally not known to the designer at the time of camber prediction. Its variability significantly impacts the value of E_{ci}.

DIFFERENTIAL TEMPRETURE AT PRESTRESS RELEASE

Concrete temperature is elevated in the first hours after concrete placement. Temperature rise is caused by the heat generated through the cement hydration process and also due to externally applied heat of curing. The temperature is higher with increased cement content. Cooling of the girder concrete to achieve balance with ambient temperature is not uniform. It is quicker in the top flange and web than in the bottom flange. The temperature gradient through the girder depth can create a deflection that is generally not considered in estimating the initial camber. This deflection component eventually diminishes. But, creep and shrinkage effects begin to take place. It is difficult to have a point in time when only elastic effects exist. In some box girder applications, the bottom flange concrete is placed first. A day later, the stems and the top flange concrete is placed. The next morning, the strands are detensioned. In this situation, the two-stage concrete placement could create differential creep, shrinkage, and temperature effects that can have a significant impact on camber at prestress release.

PRESTRESS FORCE AND SECTION

As indicated earlier, most currently used methods of camber prediction use the estimated elastic loss at midspan as a constant for the entire girder length. When prestress force, after allowance for elastic loss, along with gross concrete section properties is used, the corresponding camber would be equal to 5.39- 2.45 = 2.94 in.

DEBONDING AND TRANSFER LENGTH IGNORED

The corresponding camber would be equal to 5.36- 2.32 = 3.04 in.

FULL LENGTH VERSUS FINAL SPAN LENGTH

If camber is measured on the prestressing bed just after the release of prestress, some designers use the total length of the girder as the span length. If this assumption is used, the corresponding camber becomes 5.45-2.43= 3.02 in.

STORAGE SPAN LENGTH VERSUS FINAL SPAN LENGTH

Often the girders are placed on hardwood supports in storage at a significant distance away from the ends. They are kept in storage in this manner until they are moved for shipping to the job site. This is especially recommended for long girders to enhance their stability. Assuming the supports are 10 ft way from the ends, the corresponding camber = 4.00-1.25 = 2.75 in.

FRICTION AT GIRDER ENDS DUE TO PRESTRESS RELEASE

This is a highly variable effect. There is no guidance in the literature to follow to quantify it. It is more related to quality control issues than to true camber variability. To reduce the difference between theoretical and actual initial camber, it is recommended that the girder be lifted off the bed then reset on the bed before camber is measured.

The results of the variation in initial camber of Example 1 due to the above varying assumptions are given in Table 4. They demonstrate the importance of estimating the concrete modulus of elasticity at prestress release. Support location during storage can also have significant effects.

Table 4 Variability of initial camber of Example 1 due to some of the contributing parameters

	Camber due to initial Prestress	Deflection due to girder weight	Net	Percent change
Base line	5.33	2.32	3.01	0
High E_{ci}	4.16	1.81	2.35	-22
Low E_{ci}	6.50	2.83	3.67	22
$0.9E_{ci}$ (soft aggregates)	5.92	2.58	3.34	11
Using P_o and gross section properties	5.39	2.45	2.94	-2
Debonding and transfer length effects ignored	5.36	2.32	3.04	1
Full girder length rather than final span length	5.45	2.43	3.02	0
10 ft used for overhang length in storage	4.00	1.25	2.75	-9

BACKGROUND AND METHODS OF LONG-TERM CAMBER PREDICTION

The PCI Bridge Design Manual (2003) has a detailed discussion in Sections 8.7 and 8.13 of both approximate and detailed methods of time dependent analysis for prestress loss, camber, and deflection. The discussion includes the simple constant multiplier method originally proposed by Martin in 1977. This method is still predominant in commercial software 31 years later due to its extreme simplicity and the belief that it is difficult to accurately predict time dependent effects. Some believe that if modulus of elasticity, creep, and shrinkage can only be predicted within ± 20 percent at best, one should not worry about fine tuning the camber equations.

However, as can be seen in the following discussion, one should never worry about the random variables that cannot be controlled. Errors from controllable and/or known variables can and should be minimized as much as possible.

Due to their historical significance, the source of Martin's multiplier will be explained. This will allow for determination of whether they are valid considering the technological advances made with high performance materials and high levels of prestress.

According to Martin, to obtain camber due to prestress at time of erection (introduction of "topping" weight) the elastic camber is multiplied by a constant of 1.80. The corresponding constant for self weight deflection is 1.85. Martin started with acceptance that for cast-in-place reinforced concrete, the ACI 318-71 Code specified a creep multiplier of 2.00. The multiplier is modified to account for compression reinforcement. The modifier has changed over the years, but, the creep coefficient has remained constant as of the ACI 318-08 Code. Martin contended that prestressed concrete is originally loaded when the modulus of elasticity is lower than that at service. He estimated the ratio E_{ci} / E_c is equal to 0.85, corresponding to a compressive strength ratio of about 0.70. Thus the creep coefficient for precast prestressed members should be 0.85*2.00=1.70 to account for the fact that a low modulus is used to calculate elastic camber and deflection at prestress release. Thus, the elastic plus creep effects due to self weight at final equal 1.00 +1.70 = 2.70. The multiplier for prestress is derived the same way, except that a reduced prestress of =85% of the initial prestress is used to account for long term prestress loss. Thus, the creep multiplier is 0.85*1.70 = 1.45. The elastic plus creep multiplier is (1+1.45) =2.45. For superimposed dead loads introduced at time of erection, e.g. deck weight, the multiplier is simply = 1.00+2.00 = 3.00, as the elastic deflection is calculated with the modulus of elasticity at final conditions.

For camber at erection, Martin estimated that only 50% of the creep develops at time of erection. No effect of load duration was taken into account. Thus, the creep multiplier due to self weight at erection is 0.5*0.85*2.00= 0.85, and the total multiplier is 1.85. For the upward component due to prestress, the "average" prestress force is taken as the average of 1.00 and 0.85 times the initial prestress. That average is (0.85+1.00)/2= 0.925 and the creep multiplier is then$0.85*0.925 \approx 0.80$. Thus, the total elastic plus creep multiplier of prestress is 1.00+0.80 = 1.80.

Although this paper is limited to conditions at erection, it is worth noting that the coefficients for composite members in Martin's method are based on hollow core and double tee building members with 2 in. concrete topping. In these applications, he estimated that the ratio of girder moment of inertia to composite section moment of inertia is approximately 0.65. This obviously is much different from conditions for I-girder bridge systems. It should also be noted that the differential creep and shrinkage between the girder and the deck were ignored. Therefore, one should be extremely cautious in considering the Martin multipliers to determine final long term camber/deflection. Fortunately, such parameters are only important in design when there is concern about too much deflection causing infringement on the overhead clearance under the bridge, or causing an aesthetically undesirable sag.

CONSPAN and PSBeam are the most widely used commercial programs in the US. Both programs have adopted Martin's multipliers.

Tadros et al. in 1985 published a paper on the topic in which multipliers are given in terms of creep variability. Also, prestress loss is calculated and accounted for separately. The contents of the paper were extensively covered as the "Improved Multiplier method" in Section 8.7.2 of the PCI Bridge Design Manual. This method was further advanced in the study for NCHRP, Project #18-07, which was adopted in the 2005 Interim AASHTO LRFD Specifications and covered earlier in this paper. The variable multiplier method allows for adjustment due to high strength concrete and high levels of prestress as currently used in bridge practice. High strength concrete can cause significant reduction in the creep coefficient. However, high strength concrete also allows for use of high prestress levels. Thus, in current practice, it is observed that initial camber can be much higher than what existed a decade ago. But, camber growth, as a percent of initial camber, is somewhat slower. Also, high strength concrete tends to undergo most of its creep in the first several months, as opposed to slow developing creep in lower strength concrete. For these reasons, it is important to accurately model modulus of elasticity, creep and, to a lesser degree, shrinkage in order to obtain reasonable camber averages.

PROPOSED LONG-TERM CAMBER PREDICTION

The values of the long term cambers and deflections can be simply obtained by multiplying the corresponding initial (elastic) values by a creep multiplier. If the action that causes creep is applied instantaneously and kept constant with time, the creep deflection is the initial deflection times the creep coefficient. This case applies to all dead loads. Prestress may be assumed to consist of two components: initial prestress which may be assumed constant with time, and long term prestress loss which may be assumed to gradually develop with time. For the latter "loading", the creep effect is reduced to 0.7 times the creep coefficient. The 0.7 factor is called the aging coefficient and is assumed constant here due to its minor impact on camber analysis. If precise time-dependent analysis is required, a variable aging coefficient may be used, from Dilger (1982), for more accurate values. Another gradually developing effect is the differential creep and shrinkage between the deck and the precast girder, once the system becomes composite. However, these effects are not considered in this paper as the focus is on the camber and deflection at prestress release and at the time of deck placement. Thus the multipliers for initial plus long term are $(1+\psi)$ for constantly sustained loads, and $(1+0.7*\psi)$ for gradually introduced loads, where ψ is the creep coefficient as calculated from Equation 16.

The elastic portion of the deflection due to prestress loss between the time of prestress release and the time of deck placement requires knowledge of the long term loss for that time period. The AASHTO Specifications detailed loss method gives formulas for prediction of that loss in Section 5.9.5.4.2. Once the long term loss is found, the deflection due to the loss should be determined in proportion to the camber due to initial prestress, as will be demonstrated in the example.

The creep coefficient for a loading applied at concrete age of t_i (days) and sustained for a duration of t (days) is:

$$\psi(t\ ,t_i)=1.9*\left[1.45-0.13*(V/S)\right]*\left[1.56-0.008\ H\right]*\frac{5}{1+f'_{ci}}*\frac{t}{61-4*f'_{ci}+t}*t_i^{-0.118} \qquad (16)$$

Where V/S = the exposed volume to surface ratio (in.). It may be approximately taken = (web width/2) for I beams and H = relative humidity (%) of the ambient air surrounding the bridge.

EXAMPLE 2

Given: The Florida 72" deep bulb tee of Example 1. The deck is assumed to be placed at 120 days after the girder is prestressed.

Required: camber just before time of deck placement.

The following parameters are input into Equation (16): V/S = 6.5/2 = 3.25 in., H = 75%, f'_{ci} =6 ksi. Loading age t_i = 0.75 day and loading duration t= 120-0.75 = 119.25 days. Thus

$$\psi(120,0.75) = 1.9 * [1.45 - 0.13 * 3.25] * [1.56 - 0.008(75)] * \frac{5}{1+6} * \frac{119.25}{61 - 4(6) + 119.25} * 0.75^{-0.118}$$

$$\psi(120,0.75) = 1.9 * [1.0275] * [0.96] * 0.714 * 0.763 * 1.035 = 1.02$$

The multiplier for initial prestress plus self weight is $(1 + \psi(120,0.75)) = 2.02$, and for the prestress loss is ($(1 + 0.7 * \psi(120,0.75)) = 1.72$.

The long term prestress loss due to creep and shrinkage of concrete and relaxation of the prestressing steel should be determined according to the detailed method of AASHTO LRFD 2007. The spreadsheet Prestress Loss_PCI BDM 9.4 070320, published on the website www.structuresprograms.unomaha.edu may be used for this purpose. The relevant results are summarized here for the readers' convenience:

Table 5 Prestress losses according to AASHTO LRFD 2007 detailed method

Elastic loss due to prestress plus self weight (Loss)	18.42 ksi
Total long-term (Initial to deck placement)	21.85 ksi

It is reasonably accurate to assume that the curvature due to prestress loss has the same distribution function along the span and to assume that the loss at midspan is the dominant factor in deflection calculations. Thus, the "elastic" deflection due to the long term prestress loss between prestress release and time of deck placement may be conveniently estimated as

$$\Delta_{el,loss} = \Delta_{ip} * (\Delta f_{lt} / f_{pi}) \quad (17)$$

Thus $\Delta_{el,loss} = 5.33 * (21.85 / 202.5) =$ 0.58 in.

Please note that this is a theoretical component that never exists alone in real life, much like separation of the initial camber due to prestress and deflection due to member weight.

What the member will experience is a net long term camber due to prestress (including long term loss) and due to self weight.

The net long term camber before deck placement is thus:

Δ_{lte}= (5.33-2.32)(2.02)-0.58*1.72= 5.09 in.

For comparison purposes, long term cambers at erection according to CONSPAN are as follows:

Camber at erection due to initial prestress (including allowance for prestress loss is 6.153*1.80 = 11.075 in.
Deflection at erection due to self weight: 2.71*1.85 = 5.014 in.
Net long term camber: 11.075-5.014 = 6.06 in.

VARIABILITY OF LONG-TERM CAMBER

There are a number of causes of variability of camber growth between the time of prestress release and the time of erection. Time of erection is defined here as the time at which the deck placement operation is completed. After that time, the barrier railing is constructed and additional overlays (if any are placed). These dead loads are applied to the composite girder-deck system. Some designers include them when determining the final roadway profile and the amount of vertical clearance below the bridge. There is inconsistency in the literature on how to handle these dead load effects. They are a small portion of the total dead load and are also introduced to a significantly larger cross section than the precast girder alone. Furthermore, it is more convenient to measure elevation of the girder soffit just after deck placement and to compare it to theoretical prediction. While dead load on the composite system is not included in the discussion in this paper, the reader is being made aware of these effects, especially when comparing prediction methods.

Rosa et al (2007) have recently completed a study at the University of Washington for the Washington DOT, on "Improving Predictions for Camber in Precast Prestressed Bridge Girders." The study included analysis as well as field measurements. It concluded with a recommendation to endorse the new AASHTO prestress loss provisions, in order to directly account for loss effects on camber, and to apply modification factors to the modulus of elasticity and creep multipliers in AASHTO to reflect conditions in the state of Washington related to the environment and to local materials. The recommended factors are 1.15 for modulus of elasticity and 1.4 for creep. The report makes an important observation of the impact of support conditions. It was observed that when the temporary camber control top strands are detensioned the camber change was different for different supports. The camber was 41 to 46 percent smaller with temporary oak blocking than with the permanent elastomeric bearing pads.

Some of the sources of variability of the long term camber component at erection follow:

VARIABILITY OF INITIAL CAMBER

Because long term camber is the product of initial camber and long term multipliers.

ACCURACY OF LONG-TERM MULTIPLIERS

Martin's multipliers are simply 1.85 and 1.80 for girder weight and initial prestress. They were derived from assumption that the creep coefficient is a constant = 2.00, with additional constants to account for prestress loss, change in elasticity modulus and partial development at an intermediate time. Proposed multipliers separate these effects and allow for actual conditions to be incorporated. Even with this refinement, concrete properties are random variables and cannot be deterministically accounted for in calculations. In the NCHRP 18-07 study, creep was reported to have less scatter with the proposed method, which was adopted by AASHTO in 2005, than with previously reported methods. Even with the improved prediction accuracy, Figure 4 illustrates that most of the experimental data points fall within 25 percent bounds. The NCHRP 18-07 study did not come to a specific recommendation for upper and lower bound values as was done for the modulus of elasticity, due to lack of data for high strength concrete creep at the time of the study. It is reasonable at this time to assume these bounds to be ± 25 %.

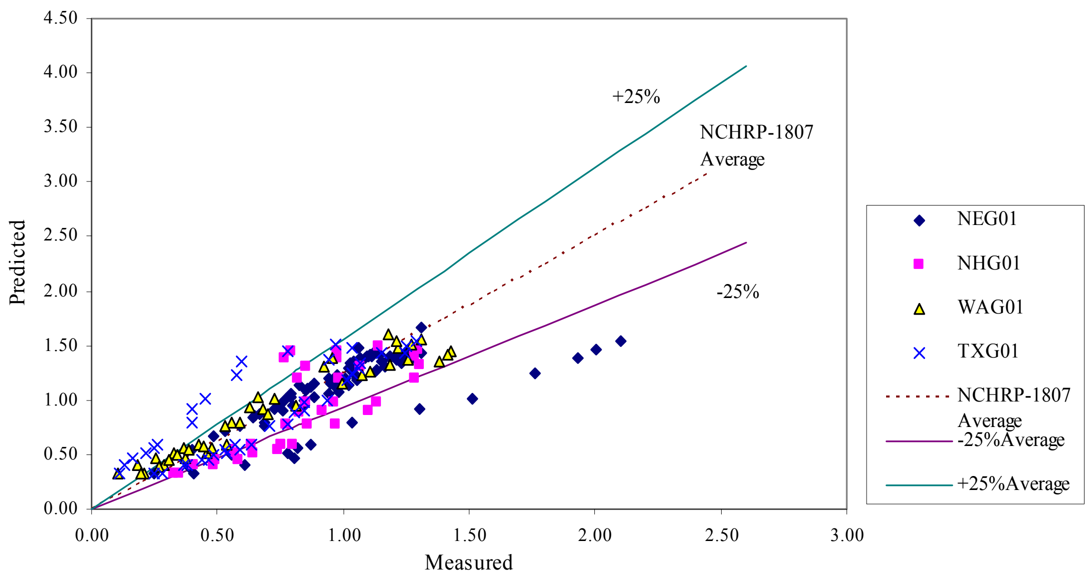


Fig. 4 Variability of creep coefficient according to NCHRP 18-07 study

ACCURACY OF PRESTRESS LOSS ESTIMATES

The detailed NCHRP 18-07 method of prestress loss prediction allows for calculation of the loss that occurs before deck placement. The previous AASHTO detailed loss prediction method only provided estimates for final loss at time infinity. Commercial software is slow to adopt the new loss method. Fortunately, the error is not significant if the total loss is used in lieu of the loss to erection time. It is not accurate, however, to assume that only 60% of the loss takes place by erection time, as was suggested in Tadros et al (1985). Recent studies shows that the long term loss after composite action has taken effect are less than 10% of the total loss

GIRDER SUPPORT CONDITION WHILE IN STORAGE

As shown before, this factor impacts the initial camber, and thus the long term camber

TIME ELAPSED BEFORE GIRDER INSTALLATION

Most designers do not, and cannot enforce a specific girder age at the time of deck placement. At best, some state highway agencies require that the girder be at least 28 days old when the deck concrete is placed. Few require 90 days. In emergency replacement cases, girders as young as several days have been installed. None of the specifications, to the authors' knowledge require an upper limit on girder age at time of deck placement. It is possible that the girders will be 6 months old before they are erected. While this variability is outside of the control of the designer, construction documents could be prepared to minimize conflicts and delays during construction. This point will be further discussed later in the paper.

DIAPHRAGM CONFIGURATION AND VARIABILITY OF TIME BETWEEN DIAPHRAGM PLACEMENT AND DECK PLACEMENT

If a rigid diaphragm encases the ends of girders from adjacent spans, then further end rotation and camber may be greatly inhibited. This situation is quite common in snow-belt regions where "integral pier" details are common to avoid salt and moisture leakage into the supports.

To illustrate the impact of some of the sources of variability mentioned above, consider an analysis by the proposed method. Two "extreme" cases will be considered:

Case A: the girders are stored on two supports at 10 ft away from the ends, deck is placed at 28 days, Eci is 22% larger than predicted, and creep coefficient is 25% smaller than predicted.

Case B: the girders stored on two supports at 9 ½ in. away from the ends, deck is placed at 180 days, Eci is 22% larger than predicted and creep coefficient is 25% smaller than predicted.

Applying the proposed method procedure, the results are summarized in Table 6.

Table 6 Long term camber variability at erection for the data of Example 1

	Base-line	Lower-bound	Upper-bound
E_{ci}, ksi	7626	5644	3608
L_o, ft	0.7915	10	0.7915
t	120	28	180
Creep	1.06	0.765	1.275
Long-term multiplier (for weight and initial prestress)	2.06	1.77	2.28
Long-term multiplier (for prestress loss)	1.74	1.54	1.89
Elastic loss	18.42	16.8	22.65
Long-term loss	21.85	8.72	31.15
Elastic camber due to initial prestress	5.33	3.35	6.6
Long-term camber due to initial prestress	10.77	5.91	15.02
Elastic deflection due to girder weight	2.32	1.03	2.92
Long-term deflection due to girder weight	4.69	1.82	6.64
Elastic deflection due to prestress loss	0.58	0.16	1.02
Long-term deflection due to prestress loss	0.99	0.25	1.92
Net long-term camber at erection	5.09	3.85	6.45

DEFLECTION DUE TO DECK WEIGHT

Using the same formula as for the deflection due to self weight, deflection due to additional dead loads on the precast member can be estimated. These additional loads include the deck weight, the weight of the cast-in-place concrete haunch (build-up) between the top flange and the deck, the weight of intermediate diaphragms, if any, and the weight of deck forms. For this example, that load is given as 1.18 kip/ft.

$$\Delta_d = \frac{5L^2}{48E_c I_{tf}}(0.1M_{e1} + M_c + 0.1M_{e2}) \tag{18}$$

Note that the modulus of elasticity and the corresponding transformed section moment of inertia correspond to the concrete age at time of deck placement.

EXAMPLE 3

The dead load acting on the precast section in addition to its own weight consists of the weight of an 8 in. deck slab (0.81 kip/ft) plus ½” additional sacrificial thickness (0.051 kip/ft) plus forms (0.082 kip/ft) plus cast-in-place haunch build up over the girder flange (0.238), totaling 1.181 kip/ft.

Substituting into Eq.18 with E_c =5,506 ksi, I_{tf} = 686,723 in^4, M_{e1}=M_{e2}=-4.44 kip-in. and M_c=32,521 kip-in. the resulting deflection = 2.37 in.

According to CONSPAN, deflection due to the same effects = 2.68 in.

NET CAMBER AFTER DECK PLACEMENT

Net camber including deflection due to superimposed dead load on precast section = $\Delta_{lte} + \Delta_d$ = 5.18-2.37 = 2.81 in.

According to CONSPAN, deflection due to the same effects = 6.06-2.68 = 3.38 in.

VARIABILITY OF DEFLECTION DUE TO SUPERIMPOSED DEAD LOADS

Superimposed dead loads acting on the precast section include diaphragms, temporary bracing, and haunch build-up over the girder top flange, deck weight, and weight of deck forms. Whether the forms are permanent stay-in-place metal forms or temporary wood forms, their weight should be included in predicting net camber immediately after deck placement.

Sources of variability of deflection include the following:

MAGNITUDE OF SUPERIMPOSED DEAD LOAD

The diaphragms, forms, and deck weight can be assumed relatively accurately. The haunch build up can be a significant load that is a function of the camber itself. Unit weight of concrete containing normal-weight aggregates is assumed 0.15 kcf. This is reasonable and consistent with AASHTO provisions for concrete strength in the 4 to 5 ksi range.

ESTIMATE OF MODULUS OF ELASTICITY E_C

This parameter is subjected to the same $\pm 22\%$ random variability discussed earlier.

SUPPORT CONDITION

This is perhaps the most significant source of variability, especially with integral abutment/pier details, see Fig. 5. When the deck is placed, the diaphragm is partially in place and already hardened. Although the diaphragm/girder connection is generally designed for no negative moment in the girder, the girder ends are in fact restrained against rotation by the diaphragm. The girders are not simply supported as assumed in deflection analysis. In the threaded rod continuity system introduced in Nebraska and elsewhere in the past 7 years, the girders are intentionally made continuous across the diaphragm for deck weight. In any case, the support details are too complex to conveniently model in camber/deflection analysis. It is known in structural analysis that deflection of a beam fixed against rotation at its ends and subjected to uniformly distributed loads is only 20% of that of a simply supported member. The actual value should be in the range of 20 to 100 percent of the simple span analysis.

BEARING RESISTANCE

Even for simply supported girders bearing on elastomeric pads with no diaphragms, as is sometimes the practice in Texas, the shearing resistance of the elastomeric bearing would render the theoretical simple span assumption imprecise.

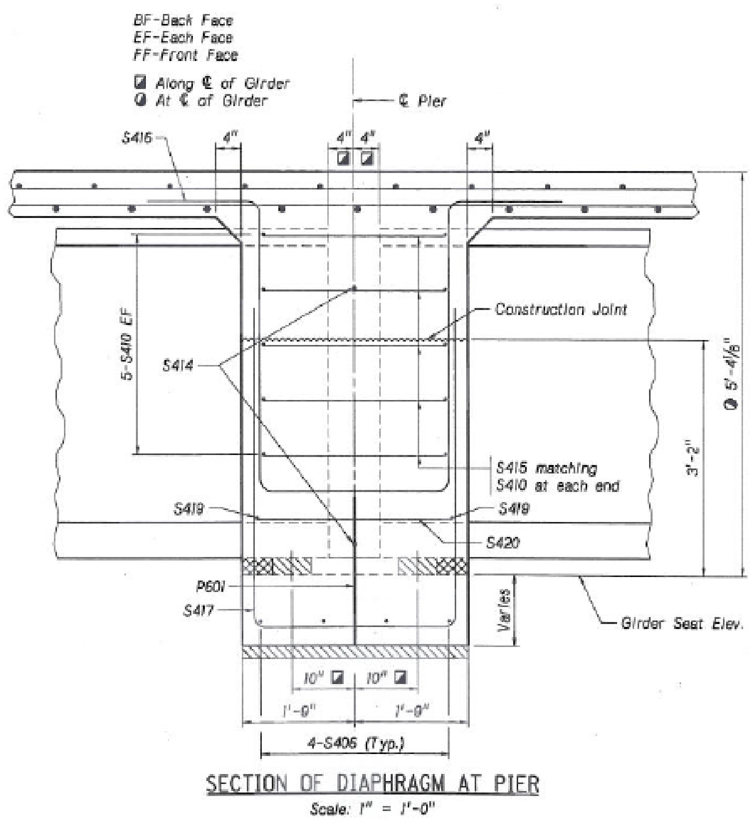


Fig. 5 Typical Nebraska Department of Roads Details of Diaphragm at Pier

DETAILING AND CONSTRUCTION CONSIDERATIONS

Assuming that the best possible preconstruction data, assumptions, and camber prediction theory are employed, there is still a likelihood of significant variation in the camber prediction. Camber at the time of deck placement is a very important measurement during construction. Yet it may vary by as much as ± 50 percent. This range could even be larger if a girder is stored in the yard for several months, see Fig. 6, for example.

Fig. 6 Visible camber in Clarks Bridge, Clarks, NE, due to extended time between prestress release and erection

For example, if theory predicts a 3 in. camber immediately after deck placement, it could end up ranging from 1.5 to 4.5 in. If the camber is larger than predicted, the girder could have a negative haunch at midspan, or even interfere with the bottom mat of deck reinforcement. If camber is lower than predicted, it would increase the quantity of concrete in the haunch, which is often an item of contractual disagreement. Increasing the quantity also increases the load on the girder. Another important factor is the possibility of infringement on vertical clearances below the bridge. Further, camber that is too small may be a cause for aesthetic concern especially if the long term camber ends up being negative, or a downward deflection. Girder sag, while acceptable to structural engineers from load capacity point-of-view, may not be as accepted by the general public.

The following guidelines are recommended in design to alleviate some of the camber variability concerns

(1) Design for a minimum haunch over the girder of 2.5 inches. This would allow for an actual camber that is 3.5 inches higher than estimated without interfering with deck

reinforcement. For a four foot wide girder top flange, and with 2% deck cross slope, the available distance is actually 3 in. not 3.5 in.

(2) Detail shear reinforcement in girders to accommodate camber variability. Typically, the horizontal shear reinforcement is pre-bent when the girder is delivered to the site. The height is fixed at 5 or 6" above the top of the flange for typical applications. This rigid solution does not accommodate relatively large camber as the hooks must be located between the top and bottom mats of deck reinforcement to be effective. Some designers use a different hook height above the top flange in the outer quarter lengths of the girder. Again, this would presume to camber prediction to be precise. There seem to be two viable options:
   a. Keep the bars projecting from the girder straight. After the bottom deck reinforcement mat is placed, used a simple pipe tool to bend the girder bars over the deck steel. This is an effective and structurally superior method. The drawback of this option is the additional field labor required.
   b. Use loose "hat" bars to supplement the pre-installed girder bars. This solution is subject to field quality control, especially if the hat bars have to be the same length and must be tilted for locations where camber is not large in order to maintain adequate concrete cover.

(3) The height of girder seats should be finalized only near the time of girder installation. At that time, the actual girder camber can be measured and the seat elevation determined. For example if the estimated camber is 3 in. and the actual camber is 1.5 in., the seat elevations can be raised by 1.5 in. using cementitious grout, steel plate or other means.

(4) The contractor pay item for concrete quantities in the haunches could be structured in a way that it is not adjustable during construction. The contractor would have to assume the variability and account for it in the initial bid. This is a small item in the overall cost of the bridge and arguments during construction should be avoided if both parties acknowledge that the engineer estimate of haunch thickness is highly variable.

CONCLUSIONS AND RECOMMENDATIONS

It is not possible to have the actual camber at prestress release or at deck placement match engineer's estimates. Random variability beyond the control of the engineer does not allow for such precision.

(1) Existence of random variability is not an excuse for making errors in theoretically estimating average camber values based on design materials and geometric properties.

(2) The Interim 2005 to AASHTO, and later editions include prestress loss, modulus of elasticity, creep and shrinkage prediction formulas that can be effectively used to improve camber prediction

(3) Local materials properties, girder storage, and construction practices should be considered in design, as much as practical, rather than defaulting to embedded conditions in commercial design software. This recommendation may not be easy to

implement as long as owners do not have specifications that govern storage and erection conditions. Currently, there are unique practices of specific producers and contractors that cannot be regulated by the designer unless the project is a design-build one, not the conventional design-bid-build.

(4) In design, allow for variability of camber by 50%. Future research may offer refinements of this figure.

(5) Allowance in design should include flexibility in adjusting the horizontal shear reinforcement and the girder seat elevations.

NOTATION

Sign convention: The following quantities are considered positive: upward camber, downward deflection, moment causing tension in the bottom fibers, eccentricity below the section centroid, prestress loss (loss of tension in the strands or compression in the concrete).

a = length of part of girder defined by Eq. (4)
a_d = distance from member end to hold down point
a_o = modified debond length = (actual debond length + transfer length/2).
b= distance defined by Eq. (5).
c = distance defined by Eq. (6).
E_{ci} = modulus of elasticity of concrete at initial time (or prestress release), Eq. (9).
E_c = modulus of elasticity of concrete at erection, assumed to be the same as modulus at service (or final time)
e_c = strand eccentricity at the center of the member measured from the centroid of the transformed section
e_e = strand eccentricity at end of the member measured from the centroid of the transformed section
e_x = eccentricity of strand group at point of debonding, as defined by Eq. (7).
f'_c = specified concrete strength in ksi, at final service conditions,
f'_{ci} = specified concrete strength in ksi, at initial conditions
H = relative humidity (%) of the ambient air surrounding the bridge.
I_{ti} = moment of inertia of precast transformed section at time of prestress release
K_1 = correction factor for source of aggregates, assumed =1.0 unless determined by testing.
K_2 = correction factor to account for random variability of modulus of elasticity.
L = span length between supports
L_t = total member length
L_o = overhang length, Eq. (3).
M_{e1}= moment at left support, negative if overhang exists, zero if overhang ignored
M_{e2}= moment at right support, negative if overhang exists, zero if overhang ignored
M_c = mid-span moment

P_i = initial prestress force in the group of strands being considered, just before release to the concrete member
V/S = exposed volume to surface ratio (in.), may be approximately taken = (web width/2) for I beams.
W = intensity of uniformly distributed load due to girder weight
w = unit weight of concrete, Eq. (10), for normal weight aggregates, or measured for mix being used.
Δ_g = mid-span deflection due to girder self weight.
Δ_{gi} = mid-span deflection due to girder self weight at initial time, Eq. (15).
$\Delta_{el,loss}$ = elastic deflection due to long-term loss between initial time and deck placement, Eq. (17).
Δ_{lte} = net long-term camber before deck placement.
Δ_d =elastic deflection due to deck weight, Eq. (18).
Δf_{lt} = Total long-term losses (initial to deck placement), Eq. (17).
ϕ_1 = curvature due to portion of prestress with constant eccentricity, Eq. (10).
ϕ_2 = curvature due to the difference of eccentricity between the debond point and the harp point, as defined by Eq. (11).
$\psi(t, t_i)$ = creep coefficient for a loading applied at concrete age of t_i (days) and sustained for a duration of t (days), Eq. (16).

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Appendix- Camber Calculation Spreadsheet

Gross Section Properties	A =	920.7	in^2
	Ycg =	34.05	in
	Ixo =	655930	in^4

Note: Input data shown in

Full length	137.083	ft	1645	in.
Span length on the bridge	135.5	ft	1626	in.
Girder concrete strength, initial	6	ksi		
Girder concrete strength, final	8.5	ksi		
Strand diameter	0.6	in.		
Strand area	0.217	in2		
Initial Prestress	202.5	ksi		

Strand Groups		No. of strands	$y_{ps,\ center}$	$y_{ps,\ end}$	a_d	Debond length	a	
	1	33	5.97	5.97	67.75	0	0.7085	
	2	4	3	3	67.75	14	14.7085	
	3	4	5	5	67.75	8	8.7085	
	4	3	7	7	67.75	6	6.7085	
Total strands		44	5.68	5.68				
Concrete unit weight			0.1485 pcf		<0.155	OK	>0.145	OK
Modulus of elasticity	Initial		4626 ksi					
	Final		5506 ksi					
Strand modulus of elasticity			28500 ksi					
Modular Ratio	n_i		6.1612					
	n		5.1764					

$$E_{ci} = 33{,}000\, K_1 (0.14 + 0.001 f_c')^{1.5} \sqrt{f_{ci}'}$$

Transformed section properties

Initial

Component	A	y	I
precast	920.7	34.05	657842
Strands 1	36.9593845	5.97	26227
Strands 2	4.47992539	3	3927
Strands 3	4.47992539	5	3415
Strands 4	3.35994404	7	2203
	970.0	32.61	693615

Final

Component	A	y	I
precast	920.7	34.05	657207
Strands 1	29.90754	5.97	21645
Strands 2	3.625156	3	3234.9
Strands 3	3.625156	5	2816.3
Strands 4	2.718867	7	1820
	960.6	32.87	686723

Reinforced Concrete unit weight	0.1535	pcf	
Girder unit weight	0.9814	kip/ft	
Deck weight	0.810	kip/ft	
Sacrificial (1/2")	0.051	kip/ft	**1.1810**
Haunch	0.2380	kip/ft	
Forms	0.0820	kip/ft	

Camber Due to Initial Prestress

Strand grou	No. of strands	Force, kips	$L_{o,}$ ft	a_o	a	a_d	b	c	e_c	e_e	e_x	ϕ_1	ϕ_2	Δ_{ip}
		kips	ft	ft	ft	ft	ft	ft	in.	in.	in.	in^{-1}	in^{-1}	in.
1	33	1450.10	0.7915	1.5000	0.7085	67.75	66.2500	0.7915	26.64	26.64	26.64	1.20E-05	0	3.978
2	4	175.77	0.7915	15.5000	14.7085	67.75	52.2500	0.7915	29.61	29.61	29.61	1.62E-06	0	0.511
3	4	175.77	0.7915	9.5000	8.7085	67.75	58.2500	0.7915	27.61	27.61	27.61	1.5125E-06	0	0.492
4	3	131.83	0.7915	7.5000	6.7085	67.75	60.2500	0.7915	25.61	25.61	25.61	1.05E-06	0.00E+00	0.344
		1933.47												**5.325**

Deflection due to member weight

M_{e1}=	$wL_o^2/2$	3.69	kip-in
M_c	$wL^2/8-M_{e1}$	27025.55	kip-in.

$$\Delta_{gi}=\frac{5L^2}{48E_{ci}I_{ti}}(0.1M_{e1}+M_c+0.1M_{e2})$$

Camber due to prestress	**5.33 in.**
Deflection due to girder	**2.32 in.**
Net at release	**3.01 in.**

AASHTO 2007 Detailed Loss Method

Imported from Excel Spreadsheet "Prestress Loss_PCI_BDM_Ex 9.4 070320, website www.structuresprograms.unomaha.edu

Elastic loss due to initial prestress plus self weight (Loss)	18.42	ksi
Total long-term (initial to deck placemnt)	21.85	ksi

$$\psi(t,t_i)=1.9*[1.45-0.13*(V/S)]*[1.56-0.008H]*\frac{5}{1+f_{ci}}*\frac{t}{61-4*f'_{ci}+t}*t_i^{-0.118}$$

Creep Multipliers:

Initial prestress $(1+\psi)$

Dead load $(1+\psi)$

Prestress loss $(1+0.7*\psi)$

ti	0.75	Creep coef	
t	120	size factor	1.0275
V/S	3.25	humidity	0.960
H	75	strength	0.714
fci	6	duration	0.763
		ldg age	1.035
		Creep coef	**1.022**

	Initial P	Self weight	Prestres s loss
Elastic	5.33	-2.32	-0.57
multiplier	2.02	2.02	1.72
erection	10.77	-4.69	-0.99

Camber at erection	**5.09 in.**
Deflection due to deck weight	**2.37 in.**
Net after deck placement	**2.72 in.**

Deflection due to dead load on precast member:

M_{e1}=	$wL_o^2/2$	4.44	kip-in
M_c	$wL^2/8-M_{e1}$	32520.74	kip-in.