Cylinder Testing for Acceptance of Precast Prestressed Concrete Bridge Girders

Q5: *Does the standard ASTM cylinder test "match" the concrete strength in the in-place prestressed concrete member?*

A5: Designers of precast, prestressed bridges and other structures are interested in the strength of concrete at the time of prestress release, f'_{ci}, and at the time of full service load application, f'_c. Precast concrete fabricators must produce girders to meet these strength requirements. Concrete cylinder testing is generally the basis for acceptance of the strength of these members.

The preparation of a cylinder cannot duplicate the production of a precast, prestressed concrete member for the following reasons:

1. The mass of concrete in a cylinder is very different from that of a large amount of concrete in a bridge girder. The temperature increase due to heat of hydration in the two products is significantly different.

2. The consolidation of a precast, prestressed member is done through internal and external mechanical vibration, while a cylinder is manually rodded.

3. Most precast plants have outdoor prestressing beds for production of large bridge girders. The temperature and humidity variation are random and change from one season to another, and from one girder to another. During the first few months of girder curing, the concrete strength will be randomly variable until cement hydration is substantially completed.

4. The 28-day age at which cylinders are generally tested does not always represent the age at which the concrete girder is subjected to full service load.

5. A certain concrete mix design determines the potential strength level of concrete, while the curing environment determines how soon and how closely that strength level can be attained.

The age of concrete at prestress release is well defined as 16 to 24 hours. For this situation, the designer needs to know whether concrete in the member is strong enough to accept the initial prestress force. The best solution available at this time is to use a cylinder subjected to the same level of accelerated curing as the member. Until a better method, e.g., nondestructive testing of the member, is developed, match-cured cylinder testing will continue to be used.

Beyond prestress release, the main interest of the designer is whether the concrete is of adequate quality to reach acceptable strength when the member is subjected to full service loads or not. The age of concrete at the time of full service load application to the member is unknown. For this situation, the 28-day cylinder testing can only be used as a rough estimate to determine the eventual strength of the concrete mix.

Accordingly, all cylinder testing must follow the same consistent standard preparation, curing and testing procedures, such as ASTM Designation C31 and AASHTO Designation T23: Standard Practice for Making and Curing Concrete Test Specimens in the Field. However, it should be understood that the cylinder strength cannot exactly measure the strength of the in-place member. Trying to make the cylinder match the conditions of a member is a futile effort and does not serve a very useful purpose.

References

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Allowable Compressive Strength of Concrete at Prestress Release

Q6: *What is the rationale behind setting allowable compressive stress of concrete at prestress release equal to $0.6f'_{ci}$? How does concrete react to a higher stress level?*

A6: Both the ACI 318 Building Code[1] and the AASHTO Standard Specifications for Highway Bridges[2] require that concrete compressive stresses due to prestress plus unfactored service loads be limited to a fraction of the concrete compressive strength. Two concrete loading combinations need to be checked: at the time of prestress release to concrete, and at time infinity. The second loading combination consists of effective prestress, after time-dependent losses have occurred, and full dead-plus-live loads. Ref. 3 includes an extensive discussion of the allowable compressive stresses for this loading combination. As a result of that background paper, both ACI 318-95 and the AASHTO Specifications (1996) have been revised to include a higher allowable compressive stress limit of $0.6f'_c$.

The subject of this discussion is the allowable concrete compressive stress at prestress release. Increasing the allowable concrete compressive stress at release may have a very significant economic impact. Examples of potential benefits include:

1. Prestress can be released at lower concrete strength than currently permitted and thus a more rapid prestressing bed turn-over results.

2. The demand for debonding or draping of strands at member end is reduced.

3. The cost of accelerated curing may be reduced.

4. More prestressing can be introduced into a given member, thus increasing its load carrying capacity.

Both the ACI 318 Code and AASHTO Bridge Specifications set the criteria for allowable compressive stress of concrete at prestress release as $0.6f'_{ci}$, where f'_{ci} is the compressive strength of concrete at the time of initial prestress. This limit appears to be intended to guard against concrete crush-

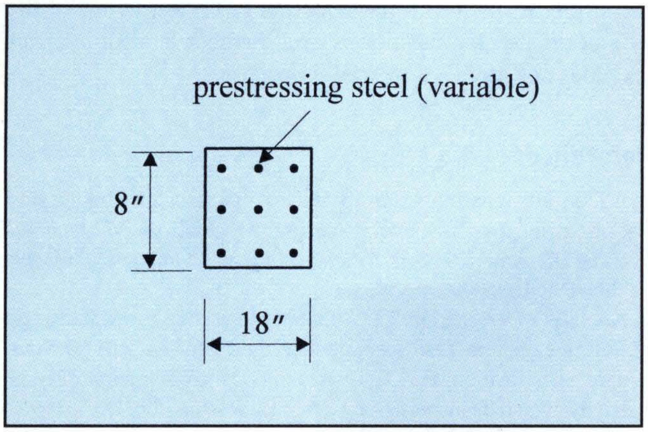

Fig. 5. Cross section of axially prestressed member example.

ing at the time of prestress release.

In order to get a better understanding of the performance of concrete at stress levels higher than $0.6f'_{ci}$, an axially prestressed member example whose cross section is given in Fig. 5 will be discussed below.

The following data are used: normal weight concrete; f'_{ci} = 3500 psi (24 MPa); E_{ci} = 3587 ksi (24732 MPa); ε_{cu} = 0.003; $^1/_2$ in. (12.7 mm) diameter low-relaxation 270 ksi (1860 MPa) strands; E_{ps} = 28,500 ksi (196507 MPa); f_{pi} (just before release) = 189 ksi (1303 MPa).

It is required to determine the concrete and steel stresses and strains due to a progressively increased number of strands. The strands will be arranged in all cases to produce a concentric prestressing force. The results of two different methods of analysis will be compared: (a) conventional linear analysis method; and (b) nonlinear analysis method.

The analysis by the two methods was performed for a range of 20 to 62 strands in the member. Details are shown below only for one case: 20 strands. This is followed by a graphical representation for the other cases and a discussion of the results.

(a) Linear elastic analysis

Linear analysis is the method commonly used in checking the allowable concrete compressive stress. The general relationship $f = \varepsilon E$ is assumed to be valid, where f = stress, ε = strain and E = modulus of elasticity.

The compressive stress in concrete after release, f_c, is given by Eq. (1) below. Refer to the notation list for definitions of the various symbols used.

$$f_c = f_{pi}(A_{ps})/[A_g + (n - 1)A_{ps}] \tag{1}$$

The corresponding concrete strain:

$$\varepsilon = f_c/E_{ci} \tag{2}$$

The steel stress after release:

$$f_{po} = f_{pi} - nf_c \tag{3}$$

Substituting for f_{pi} = 189 ksi (1303 MPa), A_{ps} = 3.06 in.^{2} (1974 mm^2), A_g = 324 in.^{2} (209044 mm^2), $n = E_{ps}/E_{ci}$ = 7.945, the following results are obtained:

$$f_c = 1.68 \text{ ksi (11.58 MPa) or } f_c = 0.48f'_{ci}$$

$$\varepsilon = 467 \times 10^{-6} \text{ or } \varepsilon = 0.16\varepsilon_{cu}$$

$$f_{po} = 177 \text{ ksi (1220 MPa) or } f_{po} = 0.65f_{pu}$$

The above solution is equivalent to the iterative approach used in some books. It is slightly more accurate than the standard 10 percent elastic prestress loss assumption used in the PCI Design Handbook.[4] By increasing the number of strands in the member, the concrete stress f_c is increased. When the member is prestressed with 45 strands, f_c becomes equal to f'_{ci} and concrete would theoretically crush. Fig. 6 shows the changes of stress and strain due to progressively increasing number of strands in the cross section.

(b) Nonlinear analysis

This analysis is based on the nonlinear concrete stress-strain relationship shown in Fig. 7 and represented by Eq.

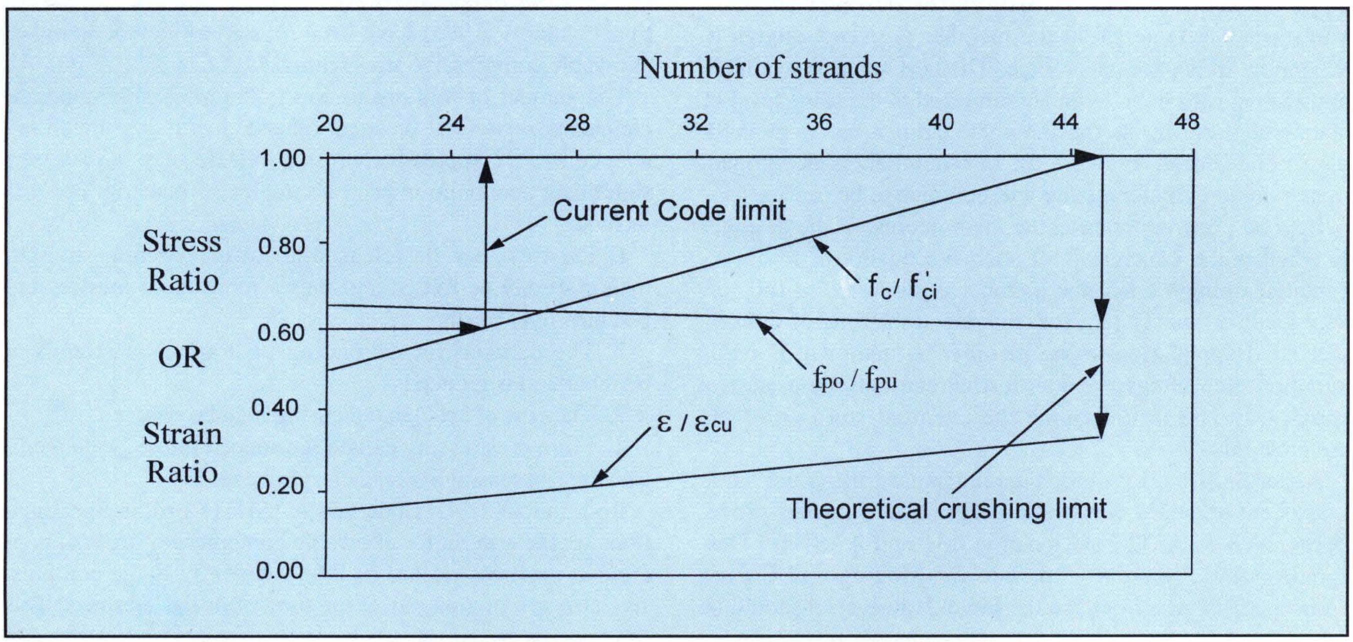

Fig. 6. Release stresses and strains in an 18 x 18 in. (457 x 457 mm) section using linear analysis.

(4), see Ref. 5. There are a number of other stress-strain models available for concretes of different age and strength. The model used here is relatively conservative. However, using other models should not significantly affect the trends or conclusions presented here. The conditions that must be satisfied are: the compatibility of strains, i.e., change in steel strain = change in concrete strain, and the equilibrium condition, i.e., concrete stress resultant = steel stress resultant.

Fig. 7 and Eq. (4) imply that concrete fails when the strain, not the stress, reaches its ultimate value. This, of course, is true in standard cylinder testing where the load is applied through controlled increments of strain. There is similarity between strain-controlled cylinder testing and a pretensioned member as the member is not expected to crush until the ultimate strain is reached. In this example, it is assumed that the ultimate strain is 0.003. In other situations, when concrete is confined by means of closed ties or spirals, the ultimate strain can be much higher. It should be noted that the extreme compression fibers in a prestressed member due to prestress release are temporary in nature as superimposed loads cause these fibers to have much reduced compression, or even tension, when the member is in service.

The concrete stress-strain relationship:

$$f_c = f'_{ci}\left[\frac{2\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o}\right)^2\right] \text{ where } \varepsilon > 0.003 \quad (4)$$

The steel stress-strain relationship:

$$f_{po} = f_{pi} - \varepsilon E_{ps} \quad (5)$$

The equilibrium condition:

$$f_{po}A_{ps} = f_c A_c \quad (6)$$

Combining Eqs. (5) and (6), the concrete stress after release:

$$f_c = A_{ps}(f_{pi} - \varepsilon E_{ps})/A_c \quad (7)$$

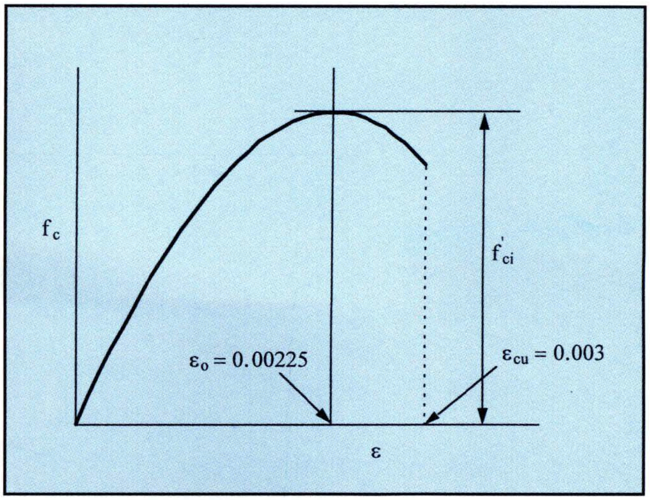

Fig. 7. Typical concrete stress-strain diagram.

Eqs. (4) and (7) can be solved for ε and f_c by iteration. A spreadsheet program was used to give the following results:

$$\varepsilon = 608 \times 10^{-6} \text{ or } \varepsilon = 0.20\varepsilon_{cu}$$

$$f_c = 1.64 \text{ ksi } (11.3 \text{ MPa}) \text{ or } f_c = 0.47f'_{ci}$$

$$f_{po} = 172 \text{ ksi } (1185 \text{ MPa}) \text{ or } f_{po} = 0.64f_{pu}$$

By using nonlinear analysis, the number of strands can be increased up to 62. Concrete would crush only when the concrete strain of the prestressed member reaches the maximum strain of the concrete. Fig. 8 shows the stress and strain changes due to varying the number of strands.

Discussion of Results

Combining the concrete stress and strain ratios from Figs. 6 and 8 into Fig. 9 gives a direct comparison between the two methods of analysis. The following observations are made:

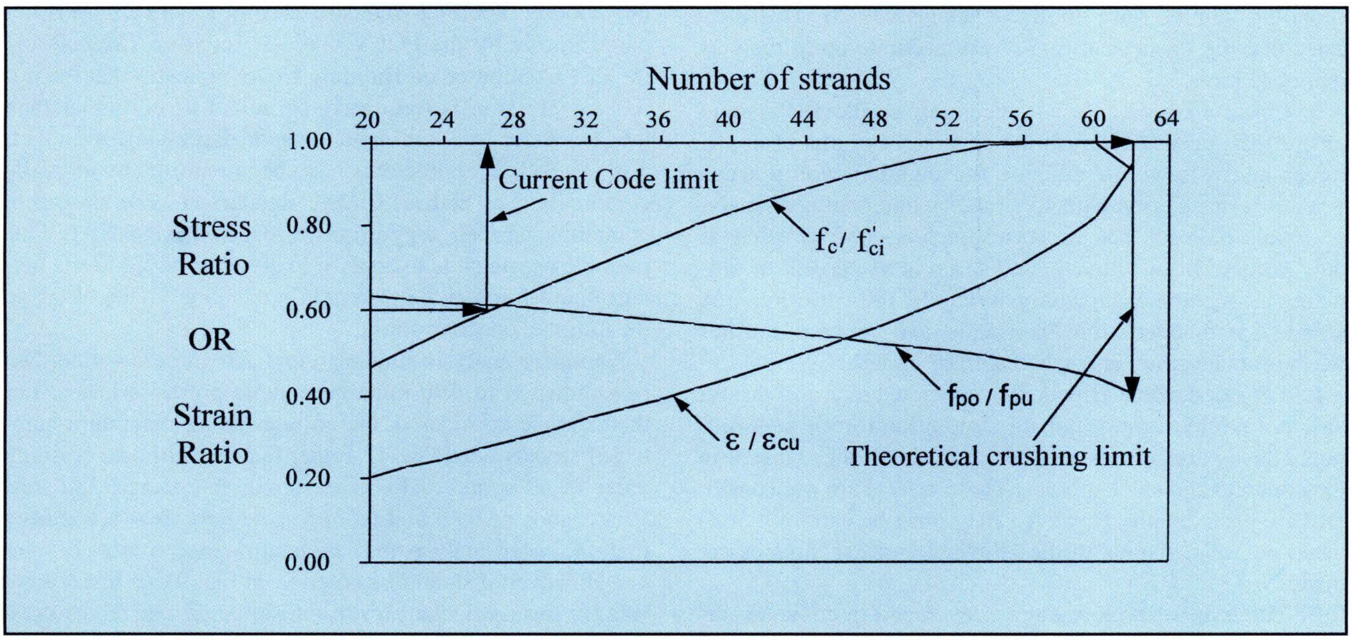

Fig. 8. Release stresses and strains in an 18 x 18 in. (457 x 457 mm) section using nonlinear analysis.

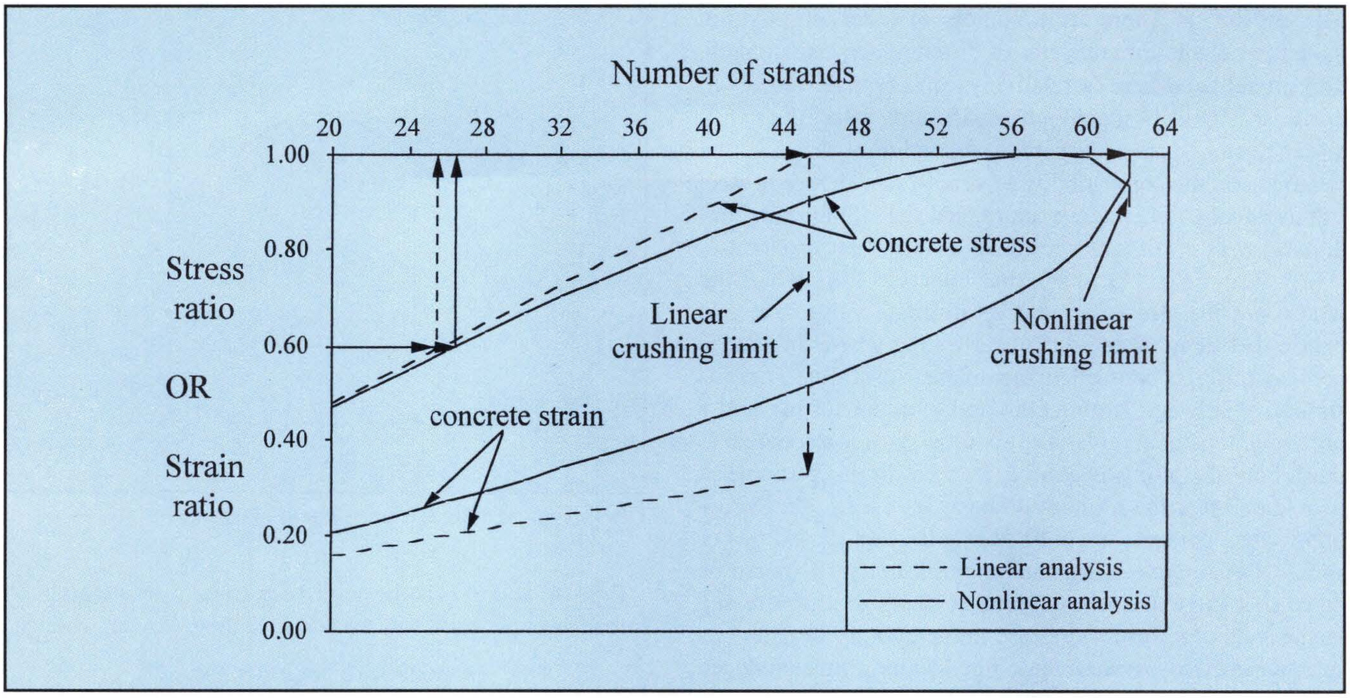

Fig. 9. Comparison between results of linear and nonlinear analysis.

1. Because pretensioning induces an internal set of stresses in steel and concrete, the behavior of a prestressed member is quite different from that subjected to externally applied compressive forces.

2. Concrete will only crush when its strain reaches ultimate strain, not when the stress reaches the peak value of the stress-strain diagram.

3. Because the system has a self-relieving mechanism, it is important to accurately account for the reduction in steel tension due to prestress release to the concrete. For example, the $0.7f_{pu}$ value before release can change to a value after release ranging from $0.65f_{pu}$ to $0.38f_{pu}$.

4. Linear elastic analysis would indicate that the code stress limit is reached when 25 strands are used. The corresponding number with nonlinear analysis is 26. This indicates that the linear analysis is very accurate up to the current code limit.

5. Linear analysis does not accurately predict the amount of prestress needed to crush the concrete. For this example, linear analysis would indicate the member would crush when it is prestressed with 45 strands while nonlinear analysis would indicate that the corresponding concrete stress is only 90 percent of ultimate, and strain is 51 percent of ultimate. Using nonlinear analysis would indicate the peak stress f'_{ci} is reached with 58 strands, and concrete crushes when ultimate strain is reached with 62 strands.

6. Time dependent effects, i.e., concrete creep and shrinkage and strength gain, during storage and until superimposed loads are introduced will result in further change in the concrete and steel stresses. These effects are not considered in this example. However, they must be carefully evaluated before revisions to the allowable release stresses are made.

7. The issue of bond capacity developed in concrete surrounding a prestressing strand may need to be re-evaluated, especially for relatively short members, before the allowable release stress is increased.

8. Another issue in need of consideration is the tolerance in strand placement and the corresponding impact of accidental eccentricity of the prestress force.

9. Confining concrete by means of spirals or closed ties has been shown to increase its ultimate strain by as much as 300 percent. This could allow for much higher prestress forces to be released without concrete crushing. This factor will need to be included in setting future criteria.

Recommendations

At this time, the authors cannot make definitive recommendations. The PCI Standard Design Practice report prepared jointly by the PCI Technical Activities Council and the PCI Committee on Building Code[6] indicates that the initial compression is frequently permitted to go higher than $0.6f'_c$ in order to avoid debonding or depressing strands. It also states that no problems have been reported by allowing compression as high as $0.75f'_{ci}$. Readers are encouraged to communicate their suggestions to the PCI JOURNAL. One possible approach is to apply a load factor of 1.2 to the pretensioning force and a "strength" reduction factor of 0.6 to the ultimate concrete strain.

Nonlinear analysis similar to that shown here would then be conducted to determine acceptable prestress levels. For the example considered, the corresponding maximum number of strands would be 42, rather than the ultimate crushing value of 62 strands. This corresponds to concrete and steel stress ratios of 0.86 and 0.55, respectively. It is recognized that nonlinear analysis may be unattractive to most designers. However, it should be noted from Fig. 9 that linear analysis for this particular prestress level conservatively overestimates the concrete and steel stresses by about 10 percent.

Notation

A_g = gross cross-sectional area
A_c = cross-sectional area of concrete
A_{ps} = area of prestressing steel
E_{ci} = modulus of elasticity of concrete at prestress release
E_{ps} = modulus of elasticity of prestressed steel
f'_{ci} = compressive strength of concrete at time of initial prestress
f_c = compressive stress of concrete
f_{pi} = stress in prestressed steel just before prestress release
f_{po} = stress in prestressed steel after elastic shortening
f_{pu} = ultimate strength of prestressing steel
n = modular ratio = E_{ps}/E_{ci}
ε_o = concrete strain at maximum concrete stress in stress-strain diagram
ε = strain of concrete
ε_{cu} = ultimate concrete strain

References

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2. AASHTO, *Standard Specifications for Highway Bridges,* 16th Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1996.
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Camber Variations

Q7: *How much camber variation should be expected in a precast concrete member?*

A7: The most significant design parameter in many precast concrete applications is the camber at the time of product installation. For systems where no cast-in-place topping is required, differential camber between adjacent elements must be minimized in order to facilitate connecting the elements and to minimize "bumps" along their connected edges. For composite construction such as I-girder bridge systems, it is important to have an accurate estimate of the camber at the time of placement of the cast-in-place deck. This camber combined with the estimated downward deflection due to deck weight are needed to determine the amount of haunch, i.e., deck slab thickening directly over the girder flange, that is needed to provide for a smooth top-of-deck profile. A smooth profile is required for the comfort of the traveling public.

Product camber at the time of its installation consists of two components: (1) initial camber, and (2) time-dependent camber. Initial camber is a function of the member weight, prestress force after accounting for elastic losses, prestress eccentricity, concrete modulus of elasticity, and moment of inertia of the cross section. Initial camber variation in the order of 25 percent is not unreasonable. One method to reduce this variation is to accurately account for the contributing parameters. For example, elastic prestress loss is a quantity that can be theoretically calculated. It does not have to be estimated at 7 to 10 percent as some designers do. Also, the change in prestress should not be ignored between strand tensioning and prestress release due to anchorage seating, temperature changes and relaxation losses.

The most significant factor affecting camber is the concrete modulus of elasticity. (Other variables such as prestress force, strand location and span length are known with a fairly high degree of precision.) Because the elastic modulus is highly dependent on local mix materials, especially the coarse aggregates, it is recommended that producers establish values of E_c from testing, and not rely on handbook and code predictions. It should be noted that a small variation in prestress camber can result in a large variation in net camber due to prestress plus self weight. For example, if self weight produces a deflection of 2 in. (51 mm), and the prestress produces a camber of 3 in. (229 mm) for a net camber of 1 in. (25.4 mm), then a 10 percent over-estimation of the 3 in. (229 mm) camber would cause the net camber to be 3.3 – 2.0 = 1.3 in. (33 mm). This is a 30 percent error.

The camber growth with time, until the girders are installed and the deck placed, is a function of the creep, shrinkage and steel relaxation characteristics. Low relaxation steel is almost exclusively used by the industry at this time. Also, shrinkage has a small indirect effect, through the prestress loss it causes. Thus, the most important parameter is creep. Creep multipliers can vary from 0.50 to 1.50 depending on the length of storage, relative humidity and other factors. Testing and actual field experience provide more reliable values than theoretical creep prediction formulas. Creep varies randomly and the best prediction would be a ±25 percent variable. This is why field measurements of camber immediately before deck placement are necessary to obtain a satisfactory top-of-the road profile.

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