OPEN FORUM
PROBLEMS AND SOLUTIONS

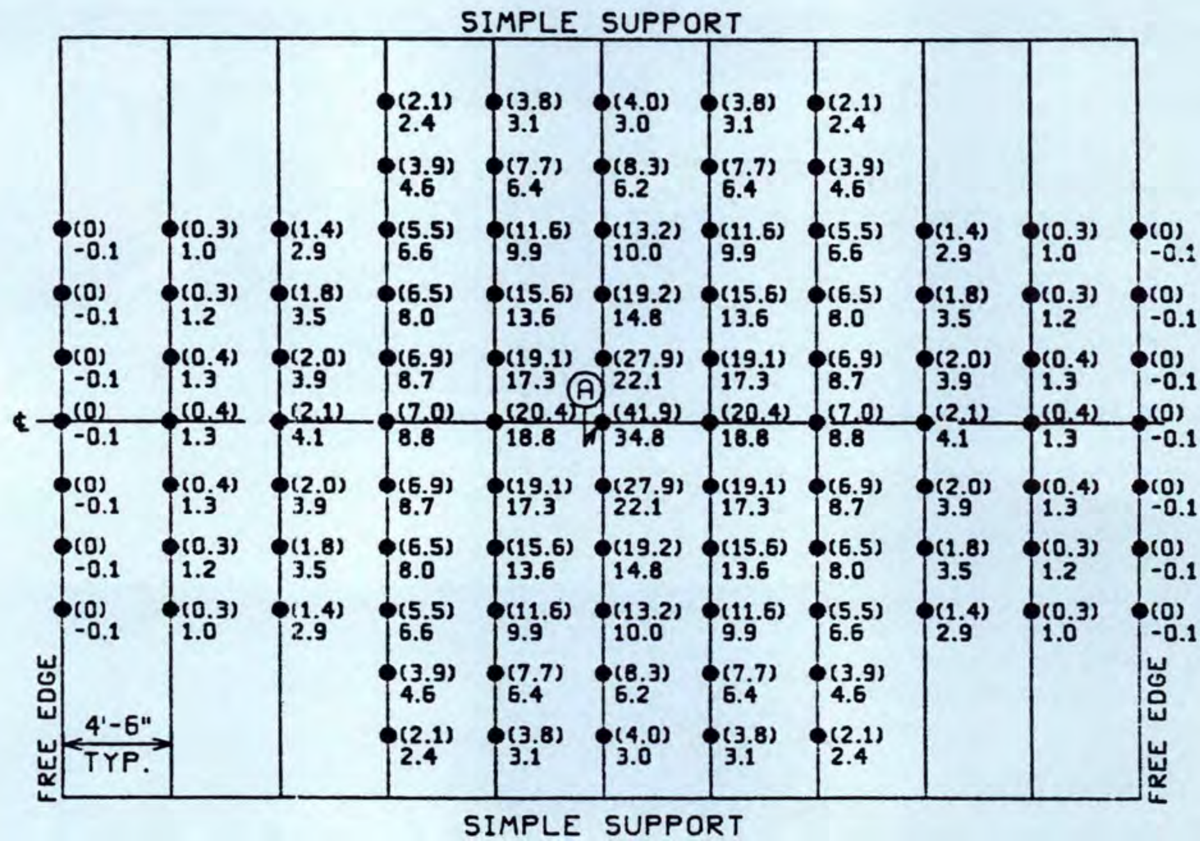
Analysis of Double Tee Floors Subjected to Fire Truck or Vehicle Loading

Q1: *How can an engineer evaluate the adequacy of a given double tee floor that is subjected to vehicular loading such as fire trucks?*

A1: This situation can arise in the event of an accidental fire in a parking structure or the more likely case of a random fire in an adjacent building. In this latter case, a moving fire truck will carry a boom and heavy eccentric outriggers.

Many past building codes and some current local codes require that driveways and sidewalks be designed for a uniform load of 250 lbs per sq ft (11.9 kPa) or for two specified concentrated loads. The loading condition producing the greater stresses must control the design.

This prescribed set of load criteria has often been extended to the case of composite double tee decks subjected to occasional fire or utility vehicles. Many of the fire trucks today are of the aerial tower variety and may weigh more than 44,000 lbs (196 kN). When the outriggers are extended and the boom is in certain positions, the load carried by a stability pad may exceed 30,000 lbs (133 kN). This is an exceptionally heavy load and the appropriate method of analysis is through the use of influence surfaces and finite element procedures.



Span = 32 ft: Influence values for moment $M_{x,a}$ (in kip-in.) in central "beam" at $^1/_2L$, in which:

$$M_{x,a} = 0.03P_iLR$$

where
P_i = load, kips
L = span, ft
R = influence value (read from chart)

Assumed model for composite section:
*Stem net depth = 23 in. E_c = 4500 ksi
*Slab thickness = 5 in. E_c = 3400 ksi
Alternately:
Stem net depth = 18 in. E_c = 4500 ksi
Slab thickness = 6 in. E_c = 3900 ksi
* Numbers on chart in parentheses

Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

Courtesy of Colorado Prestressers Association

Fig. 1. Sample values representing an influence surface for midspan moment in a deck spanning 32 ft (9.75 m).

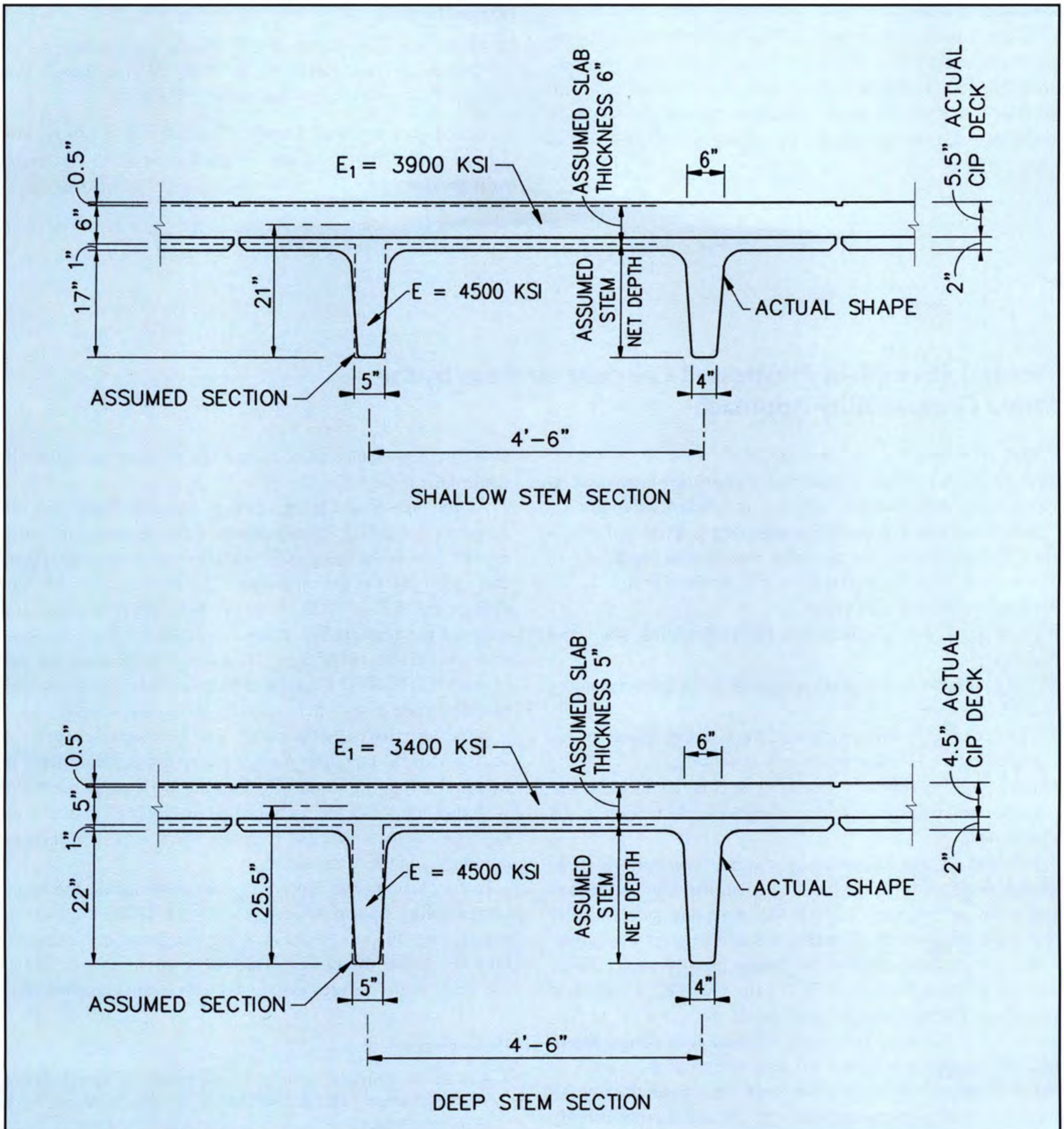


Fig. 2. Section properties and material strengths of deck spanning 32 ft (9.75 m).

A sample of values representing an influence surface for midspan moment in a deck spanning 32 ft (9.75 m) is shown in Fig. 1. The section properties and material strengths are shown in Fig. 2. In this case, the moment M_A (in ft-kips) at Point A in the central stem due to a concentrated load P_i is computed as follows:

$$M_A = 0.03\ P_i L R$$

where

P_i = concentrated load, kips

L = span, ft

R = reading on chart under load

A detailed study conducted in 1985[1] demonstrated that the equivalent uniform live load to produce the same midspan moment as a static 44,000 lb (196 kN) fire truck (with outriggers extended) varies from 163 to 88 lbs per sq ft (7.8 to 4.2 kPa) for spans ranging between 32 and 54 ft (9.75 and 16.46 m), respectively.

Although the 250 lbs per sq ft (11.9 kPa) uniform load appears to be conservative for moments, it is substantially unconservative when it comes to punching shear, stem shear design and transverse deck flexure. For the geometry shown in Fig. 2, it has been the practice since 1985 to use an approximately 5½ in.(140 mm) thick cast-in-place slab with two layers of mild reinforcing steel.

In conclusion, for strength, safety and economic reasons, the replacement of a fire truck or utility vehicle by a uniform live load should not be used. A more rational approach (such as the finite element method) is strongly recommended for generating the moment and shear influence surfaces.

Reference

1. Aswad, A., Mack, P., Hansen, R., Prebis, W., and Wathke, D., "Design of Precast Parking Decks With Fire Truck Load," Colorado Prestressers Association, Denver, CO, 1985.

[Contributed by Alex Aswad, Professor of Engineering, Civil Engineering Program, Pennsylvania State University at Harrisburg]

Flexural Strength of Prestressed Concrete Sections by the Strain Compatibility Approach

Q2: *How large is the increase in the computed moment strength by the strain compatibility approach compared to empirical code formulas? And when is it beneficial to use it?*

A2: The strain compatibility approach is a rational procedure for calculating the available moment strength, M_u, in flexural numbers. In this method, it is assumed that:

- Plane sections remain plane.
- Perfect bond exists between the reinforcing steel and concrete.
- The nonlinear stress-strain relationship for the reinforcing steel is known.
- The concrete's stress-strain curve is either given or replaced by the familiar Whitney's stress block.
- The strains in various reinforcing steel layers increase linearly (strands are not replaced by a single tendon at the centroid).

Because of non-linearities, the strain compatibility approach is not suitable for hand calculations. Many commercial software programs include the procedure automatically and solve the problem of finding the neutral axis by iteration.

The strain compatibility procedure usually yields larger computed strengths than ACI-318 or AASHTO's empirical formulas. The percentage increase in the value of M_u depends on the section geometry, whether it is composite or not, the reinforcing steel ratio and other variables such as the stress-strain curves. A pilot study[1] has shown the following additional resisting moments by the strain compatibility procedure:

- Untopped inverted T-beam: 6.6 percent
- Topped inverted T-beam: 2.0 percent
- Untopped 8DT18: 1.1 percent
- Topped 8DT14: 1.5 percent

It is also clear that the use of the strain compatibility procedure is desirable whenever ultimate strength controls. In building members such as double tees, there are no clear cut rules when this happens, though service load stresses control the majority of cases. The smaller the allowable bottom tension, the less chance there is that the ultimate strength will control the design.

As for prestressed bridge girders designed following the Standard AASHTO Specifications, ultimate strength almost never controls the design of I sections[2] (with practical spans and spacings). On the other hand, the ultimate strength will control the design of most spread-box superstructures and many of the adjacent box cases especially for allowable bottom tension stresses of $6\sqrt{f_c'}$. It is also expected that the use of AASHTO-LRFD Code[3] and finite element methods may result in more cases where ultimate strength controls.

It is worthwhile to note that the semi-empirical ACI/AASHTO equation for f_{ps}, the reinforcement stress at nominal strength, should be limited to flexural members with rectangular or tee sections that are fully tensioned with the reinforcement near the extreme fibers. Its use in piles or columns is not recommended.

In conclusion, it is always desirable to include the strain compatibility option in design software. If the engineer is conducting an investigation using longhand calculations, then the above discussion is useful in deciding whether to use iterative procedures associated with strain compatibility.

References

1. Aswad, A., "Strand Savings Using Strain Compatibility in Strength Design," PCI JOURNAL, V. 21, No. 2, March-April 1976, pp. 78-81.
2. Aswad, A., Hansen, R., and Wathke, D., *Simple Span Design Tables for Prestressed Bridges*, 3rd Edition, Stanley Structures, Inc., Denver, CO, 1985.
3. Aswad, A., and Jacques, F. J., *Review of NCHRP 12-33 Specifications for Impact on Design of Pretensioned Girders*, Precast/Prestressed Concrete Institute, Chicago, IL, October 1992.

[Contributed by Alex Aswad, Professor of Engineering, Civil Engineering Program, Pennsylvania State University at Harrisburg]

Maximum Reinforcement in Prestressed Concrete Members

Q3: *I would like to know an explanation of how to apply Section 18.8.2 in ACI 318-95[1] which states the following:*

"When a reinforcement ratio in excess of that specified in Section 18.8.1 is provided, design moment strength shall not exceed the moment strength based on the compression portion of the moment couple."

A3: This Code statement appears to contradict principles of mechanics where the equilibrium of forces dictates that the compression portion of the moment couple be equal to the tension portion. This language should perhaps be clarified by replacing "the compression portion of the moment couple" with "the maximum reinforcement ratio."

To illustrate the application of this section, let us consider the following example. An inverted tee, 40 ft (12.2 m) long, is placed on two supports 34 ft (10.4 m) apart, with a 6 ft (1.8 m) cantilever. The beam has 29 - $^1/_2$ in. (12.7 mm) - 270 ksi (1862 MPa) low relaxation strands with a draped strand pattern as shown in Fig. 3. The 28-day concrete strength is specified at 5000 psi (34.5 MPa).

The problem is to determine the negative moment design capacity of Section 2. The reinforcement index, ω_p, according to ACI 318-95, Section 18.8.1 is:

$$\omega_p = \rho_p\left(\frac{f_{ps}}{f_c'}\right) \tag{1}$$

where

ρ_p = prestressed reinforcement area ratio (= A_{ps}/bd_p)
A_{ps} = prestressing reinforcement area
b = width of compression face
d_p = depth of centroid of reinforcement from compression face
f_{ps} = reinforcement stress at ultimate flexure
f_c' = specified compressive strength of concrete

Note that f_{ps} may be obtained from Eq. (18.3), ACI 318-95, which reduces to the following equation for the reinforcement and concrete properties used in this example:

$$f_{ps} = 270\left[1 - 0.35\rho_p\left(\frac{270}{5}\right)\right] \tag{2}$$

For the reinforcement available in this section:

A_{ps} = 29(0.153) = 4.437 in.^{2} (2863 mm^2)
d_p = [7(2) + 6(4) + 16(16.8)]/29
= 10.58 in. (269 mm)
ρ_p = 4.437/(32)(10.58) = 0.0131

Substituting into Eqs. (2) and (1), f_{ps} = 203 ksi (1400 MPa) and ω_p = 0.532. Because the Code limit on ω_p is $0.36\beta_1$ = 0.288 for f_c' = 5000 psi (34.5 MPa), the section is considered by the Code to be over-reinforced.

For this case, the Code indicates that the flexural capacity corresponding to $\omega_p = 0.36\beta_1$ should be used. The flexural capacity for a given reinforcement index is:

$$\phi M_n = \phi A_{ps} f_{ps}\left[d_p - \frac{A_{ps} f_{ps}}{2(0.85) f_c' b}\right] \tag{3}$$

Replacing $A_{ps}f_{ps}/bd_pf_c'$ with ω_p yields:

$$\phi M_n = \phi f_c' b d_p^2 [\omega_p(1 - 0.59\omega_p)] \tag{4}$$

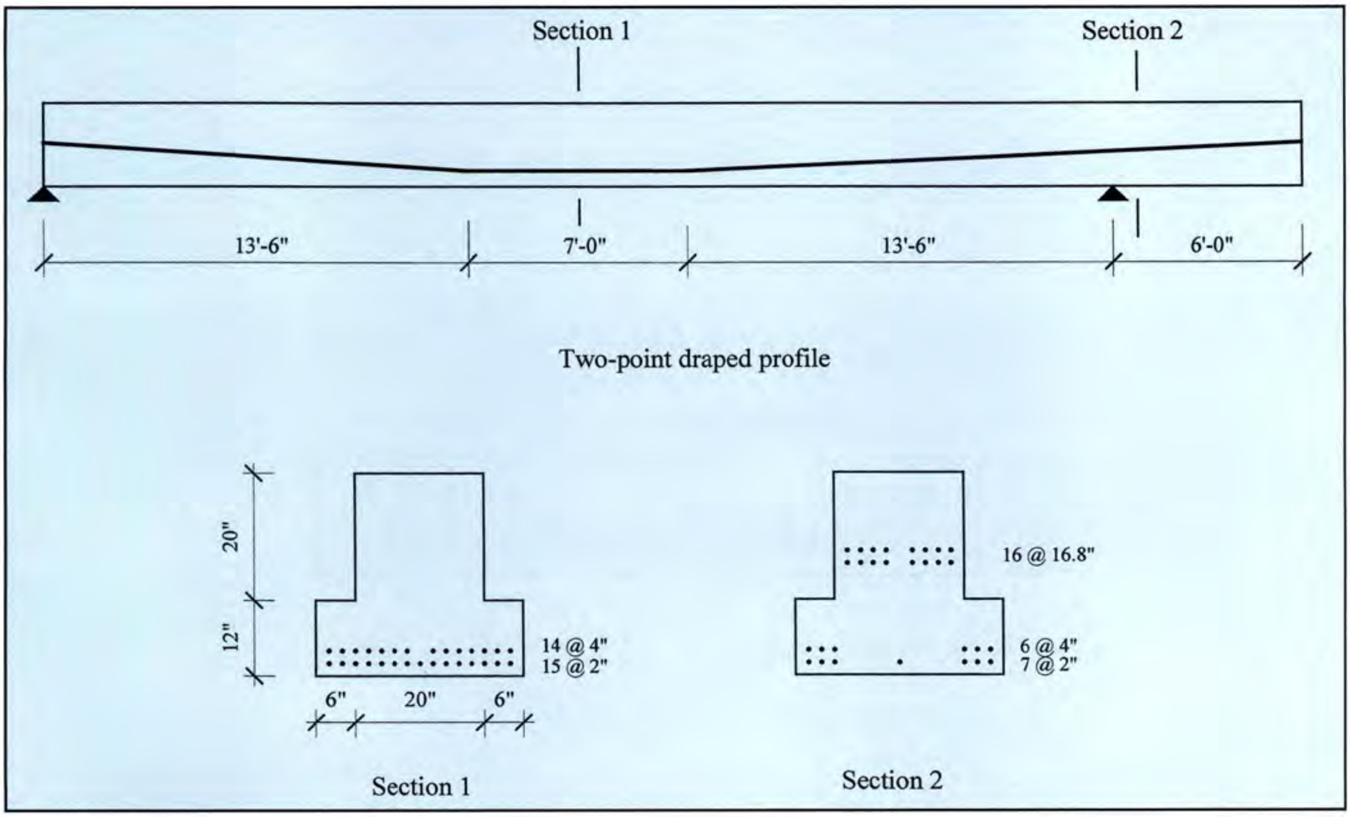


Fig. 3. Strand pattern and cross section details.

Setting $\omega_p = 0.36\beta_1$, Eq. (4) reduces to:

$$\phi M_n = \phi f_c' b d_p^2 (0.36\beta_1 - 0.08\beta_1^2) \quad (5)$$

This equation is similar to the $0.25\phi f_c' b d_p^2$, given in earlier versions of ACI 318[2] for $\beta_1 = 0.85$, which corresponds to concrete strengths of 4000 psi (27.6 MPa) or lower. Substituting into Eq. (5) with $\phi = 0.90$, $f_c' = 5000$ psi (34.5 MPa), $b = 32$ in. (813 mm), $d_p = 10.58$ in. (269 mm), and $\beta_1 = 0.80$, yields $\phi M_n = 318$ ft-kips (4.31×10^5 kN-mm).

The above discussion illustrates how to use Code Section 18.8.2. However, it will be shown below that this Code provision may not always yield realistic results.

For the example shown, let us ignore the existence of the bottom 13 strands in the calculations. In this case the revised parameters are: $A_{ps} = 2.448$ in.^{2} (1579 mm^2), $d_p = 16.8$ in. (427 mm), $\rho_p = 0.0046$, $f_{ps} = 247$ ksi (1702 MPa), and $\omega_p = 0.227 < 0.288$. Thus, the standard ϕM_n, Eq. (4), with the calculated ω_p may be used, and ϕM_n is equal to 661 ft-kips (8.96×10^5 kN-mm).

This value is 108 percent of the ϕM_n with the bottom strands not ignored. Obviously, the discrepancy cannot be theoretically justified. We are not recommending here that the bottom strands be ignored. As will be shown below using the strain compatibility approach, fully tensioned strands in the concrete compression zone may result in a small reduction of the flexural capacity.

A strain compatibility approach using the computer program developed by Skogman et al.[3] yields $f_{ps} = 239$ ksi (1647 MPa) for the top layer of strands, $f_{ps} = 101$ ksi (696 MPa) for the bottom layer of strands and nominal strength $M_n = 677$ ft-kips (9.18×10^5 kN-mm). Because f_{ps} is less than f_{py} of the top layer of strand, compression failure may take place. This is consistent with the above calculations leading to the use of Eq. (5). For this case, it may be advisable to use a lower ϕ value than 0.9 to determine the design strength ϕM_n. This is more rational than artificially limiting the steel area as implied by Section 18.8.2 of ACI 318-95. Mast's approach adopted by ACI 318-95, in Appendix B, shows a method of determining ϕ. If this approach is followed, $\phi = 0.89$ and $\phi M_n = 603$ ft-kips (8.18×10^5 kN-mm).

If the bottom 13 strands are ignored and the strain compatibility approach is used, the ϕM_n value would increase to 662 ft-kips (8.98×10^5 kN-mm), only 10 percent over the value with all strands considered. This result is more realistic than the 108 percent difference.

In conclusion, it may be stated that when the reinforcement index exceeds the maximum required by the ACI Code, it is advisable to use the strain compatibility approach with proper adjustment of the strength reduction factor ϕ in order to allow for possible compression failure as in a column. For the example given here, the strain compatibility approach gives a moment capacity of 603 ft-kips (8.18×10^5 kN-mm) compared to the Code approximation of 318 ft-kips (4.31×10^5 kN-mm).

References

1. ACI Committee 318, "Building Code Requirements for Structural Concrete, (ACI-318-95)," American Concrete Institute, Farmington Hills, MI, 1995.
2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI-318-83)," American Concrete Institute, Farmington Hills, MI, 1983.
3. Skogman, B. C., Tadros, M. K., and Grasmick, R., "Flexural Strength of Prestressed Concrete Members," PCI JOURNAL, V. 33, No. 5, September-October 1988, pp. 2-29.

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Submittals of Problems and Solutions are invited from our readers. The question-answer problems should present short, timely, practical items that are useful, educational and challenging. Each submitted item will be reviewed according to PCI policy, general interest and appropriateness.